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1.1 INTRODUCTION

This Revised Remedial Design Report (RDR) has been prepared at the request of the San Francisco Bay Regional Water Quality Control Board (RWQCB) to update the contents of the RDR issued on March 15, 2002 in compliance with Task C.3 of the RWQCB Order No 01-094: Site Cleanup Requirements for Rhodia Inc. Martinez Facility, Peyton Slough Project (the Order). This Revised RDR incorporates modifications to the engineering analysis and design derived from the ongoing permitting process and cooperation with the regulatory agencies.

As with the original RDR, this revised RDR is based on the selection of Alternative 7b- Full Re-alignment of the Slough as the preferred alternative in the Feasibility Study (FS) submitted to the RWQCB on March 2, 2001, and the subsequent Addendum to the FS and Conceptual Remedial Action Plan (AFS/RAP) submitted to the RWQCB on January 10, 2002. Alternative 7b- Full Re-alignment of the Slough includes land-based and/or barge-based excavation of a new alignment, the removal of areas of concern (AOCs) containing chemicals of concern (COCs), relocation of the tide gates into the new alignment, and capping of a portion of Peyton Slough.

This document is intended to provide a final design basis. Detailed plans and specifications may vary due to actual permit conditions, complications encountered during construction, preferred construction techniques by the selected contractor, or other unforeseen conditions.

1.1.1 Report Organization

This report is divided into six main sections. The remainder of this section includes a summary of the property ownership history and a description of the site. Section 2 presents the site regulatory history, including a summary of the RWQCB Order requirements, previous reports, the remedial action objectives, the project approach, and the proposed remedial action. Section 3 includes the engineering basis for the design. Section 4 presents the implementation plan and schedule, the conceptual wetland mitigation plan, and the long-term monitoring and post remedial controls proposed to evaluate the effectiveness of the remedial action. Section 5 provides the regulatory overview for the project and the permit status and requirements to date. Section 6 contains the list of references used in the preparation of this RDR.

To assist the reader, the following terms are used in reference to the site and project features:

- "Rhodia Property" or the "Property" refers to Rhodia's operating sulfuric acid regeneration plant.
- The term "project site" refers to the area in which remedial actions will take place and encompasses construction areas, the new alignment, the existing Slough, and the AOCs.
- The term "existing Slough" or "Slough" refers to the portion of the Peyton Slough between Carquinez Strait and the culvert at railroad tracks north of Waterfront (Marina Vista) Road. When referring to the larger slough, the term "Peyton Slough" or "slough system" is used.
- The term "AOCs" (areas of concern) generally means the existing Slough sediments and the dredge spoil piles located on the banks of the Slough.

1.2 SITE HISTORY AND DESCRIPTION

1.2.1 Property History and Ownership

Rhodia Inc. (Rhodia) operates a sulfuric acid regeneration facility located at 100 Mococo Road in Martinez, California (herein referred to as “the Property”) (see Figure 1.2-1). Rhodia’s property is comprised of approximately 114 acres immediately east of Interstate 680 on the south shore of Carquinez Strait, adjacent to the southern end of Benicia-Martinez Bridge. The Property has been in continuous industrial use since the early 1900s, and was originally owned by the Mountain Copper Company (MOCOCO). MOCOCO operated a copper ore smelter until 1923 and a pyrite roaster until 1966. Over the years, waste by-products from the smelting and roasting operations, including cinders from the roasting of pyrite ores and primary copper smelting slag, were disposed in piles on the Property. Stauffer Chemical Company (Stauffer) purchased the Property from MOCOCO in 1968, and constructed a sulfuric acid regeneration manufacturing facility, which has been in operation since 1970. Stauffer removed and sold the accumulated cinder/slag piles to various industries as raw materials. The removal of the waste piles ceased in 1976. Today, the remnants from the piles are present as underground “ore bodies” up to 40 feet below grade. In 1972, Stauffer installed a water extraction and storage system to prevent leachate from migrating out of the ore bodies. From 1989 to date, the leachate is collected and treated at the Process Effluent Purification (PEP) Plant onsite.

In 1987 Rhone-Poulenc Inc. (Rhone-Poulenc) purchased Stauffer, acquiring both the Property and the manufacturing operations. In 1998 Rhodia was created as a separate, independent corporation by Rhone-Poulenc. The Site and operations were transferred to Rhodia, which currently owns and operates the sulfuric acid regeneration and manufacturing facility.

Peyton Slough is located adjacent to the Property and continues south under Waterfront Road to McNabney Marsh (see Figure 1.2-1). A key function of Peyton Slough is to convey to Carquinez Strait storm water run-off and approximately 3 million gallons per day (MGD) of treated wastewater from Mt. View Sanitary District (MVSD), which is discharged to lower McNabney Marsh via MVSD’s marsh. Peyton Slough was originally dredged in the early 20th century as part of a mosquito abatement project commissioned by Contra Costa County to facilitate the drainage of Peyton Marsh (JRP 1997). The portion of Peyton Slough subject to this RDR (the “Slough”) is located between Waterfront Road and Carquinez Strait.

The central portion of the Slough, in the vicinity of the tide gate area, lies within the Rhodia property. The State of California owns both the northern and southern portions of the Slough (see Figure 1.2-2). There is an easement, controlled by the State of California, for the portion of the Slough running through the Shore Terminals property. The location of the new alignment is also shown on Figure 1.2-2. The relocation of the south alignment may require the purchase of land or access rights from the adjacent property owner. The relocation of the north alignment may require a land lease from the California State Lands Commission.

Some time after the MOCOCO copper smelting operation began, the original meandering slough located in the southern portion of the project site was straightened and redredged to form its current alignment (see Figure 1.2-3). Historic photographs taken between 1928 and the 1960s show that the original meandering slough continued to function and was connected to the re-dredged Slough. When the copper smelting operation ceased, the original meandering slough and other low spots on the Property were filled to grade with copper smelter slag and cinders taken from a nearby cinder/slag pile and then covered with imported fill material. The portions of the cinder/slag piles (the ore bodies) that subsided below grade were also covered in place with imported fill material.

1.2.2 Site Description

The Slough is located between Waterfront Road and Carquinez Strait and is comprised of an approximately 5,500-foot long segment of the north-flowing Peyton Slough (see Figure 1.2-3). The Slough bottom elevation is at approximately –3.5 feet National Geodetic Vertical Datum 29 (NGVD), and the embankments adjacent to the Slough are as high as +5 feet NGVD. The entire Slough has been dredged repeatedly in the past. Dredge spoils were placed along both banks of the Slough in linear piles of unknown thickness. Currently, portions of the piles rise to +7 feet NGVD (north of the tide gate) and + 4.5 feet NGVD (south of the tide gate), and some remain without vegetation. The unvegetated portions range in size and are located directly adjacent and parallel to the Slough. For the purpose of this report, the Slough has been subdivided into the “north Slough” and the “south Slough,” which are separated by a tide gate and levee. The project site layout is shown on Figure 1.2-3.

The north Slough is approximately 2,350 feet long, is generally 30 to 60 feet wide, and extends from Carquinez Strait to the tide gate. In this area, the tide generally fluctuates between +2.69 feet NGVD Mean High Water (MHW) and –0.92 feet NGVD Mean Low Water (MLW) (see Table 1.2-1). At low tide, the minimum water depth is approximately 2 feet. Most of the north Slough embankment slopes range from near vertical to 3:1, and are approximately 3 to 5 feet high. The east and west embankments of the north Slough are densely vegetated. In many locations, the embankments become inundated due to their low elevation, and the water line extends up to approximately 30 feet into the vegetation. Several tributary sloughs intersect the eastern embankment of the existing Slough. A large marsh occupies the area east of the north Slough, which is virtually inaccessible by vehicle. Only a small portion near the Rhodia polishing pond on the eastern side of the tide gate is accessible by vehicle.

The south Slough is approximately 3,150 feet long, and extends from the tide gate to Waterfront Road (see Figure 1.2-3). The south Slough width averages approximately 50 feet, ranging from less than 10 feet wide at the southern end near the culvert under the railroad embankment to approximately 60 feet near the tide gate. The tide gate is currently operated to allow freshwater to flow downstream through the gate during low tidal periods and to prevent tidal waters, which would inundate the south Slough area, from moving upstream. Tide gate operations are controlled by Contra Costa Mosquito and Vector Control District (CCMVCD). During the rainy season, surface runoff tends to inundate portions of the south Slough area. Oxbows from the former meandering slough (circa 1960) appear to intersect the south Slough in four locations. In this document, the former meanders are referred to as the “paleo-channel”. Zinc Hill is a predominant topographic feature located east of the south Slough (Figure 1.2-3). The toe of Zinc Hill is within 150 feet of the east bank of the Slough in some locations.

The levee is oriented roughly east to west and extends between the tide gate and Zinc Hill. The levee was originally constructed over an old dike in the early 20th century; however, it was raised by CCMVCD in 1952 (JRP 1997). The design elevation of the levee is +7 feet NGVD. The general elevation of the project area differs greatly between the north and south sides of the levee. North of the levee, the average elevation of the plain is approximately +3 feet NGVD;

whereas south of the levee, the elevation is approximately +0 foot NGVD. Because of the distinct elevation differences, CCMVCD uses the tide gate to restrict tidal action into the south Slough area, therefore preventing long periods of inundation.

The tide gate structure is located approximately 2,350 feet from Carquinez Strait and is approximately 67 feet wide by 37 feet long. The tide gate was installed in 1998 by the CCMVCD as a control measure to reduce flooding south of the levee (Rhodia, South Peyton, and McNabney Marshes) and to introduce tidal exchange into that area. The embankments in the immediate vicinity of the tide gate are between +5 feet NGVD and +10 feet NGVD and are protected with riprap which is up to a 2 to 3 feet in diameter.

Rhodia's polishing pond and three CCMVCD drainage ditches are located on the east side of the Slough near the tide gate (see Figure 1.2-3). The ditches branch off from a tributary of the Slough near the tide gate. The northernmost and the southernmost ditches are approximately 6 feet wide where they intersect temporary bridges and trails. The center ditch is approximately 15 feet wide.

The Slough enters Carquinez Strait approximately 1,400 feet to the east of the Benicia-Martinez Bridge. Drainage from the Sacramento and San Joaquin rivers flows through the strait into San Pablo Bay, San Francisco Bay, and ultimately the Pacific Ocean.

1.2.3 Vegetation Species at the Site

In this RDR, the distinct portions of the Peyton Slough and Marsh system have been referred to, as follows (see Figure 1.2-1):

- North Peyton Marsh: located north of the levee
- South Peyton Marsh: located east of the existing Slough, between the levee and Waterfront Road
- Rhodia Marsh: located west of the existing Slough and south of the Rhodia Property
- McNabney Marsh: located south of Waterfront Road

Bulrush (*Scirpus acutus*) and narrow-leaved cattail (*Typha angustifolia*) make up the dominant vegetation along the channel banks in the north Slough area (Fig. 1.2-4, Table 1.2-2). Portions of the dredge spoil pile embankments are vegetated, whereas others parts are devoid of vegetation due to elevated metal concentrations. The vegetated portions have an upland plant community dominated by coyote brush (Table 1.2-2). The marsh plain east and west of the north Slough is functioning tidal wetland with a variety of plant community types. The most abundant plant species include narrow-leaved cattail, alkali bulrush (*Scirpus robustus*), pickleweed (*Salicornia virginica*), and pepperweed (*Lepidium latifolium*). The dominant vegetation along the shoreline of Carquinez Strait in the project area is bulrush and common reed (*Phragmites australis*).

The banks of the south Slough are lined with an emergent vegetation community comprised predominantly of California bulrush (*Scirpus californicus*), cattail (*Typha* spp.), and three-square bulrush (*Scirpus americanus)*. Other species associated with the Slough's hydrology include common reed, Baltic rush, and tall flatsedge (*Cyperus eragrostis*). Seasonal wetlands in the project area extend east and west of the south Slough. Much of the seasonal wetlands south of the levee have limited function and value. These areas have been degraded due to lack of tidal action, subsidence, and contamination. Plant cover and distribution are influenced by contamination, microtopography, and hydrology. The following species are common in the seasonal marsh areas: saltgrass (*Distichlis spicata*), fat hen (*Atriplex triangularis*), rush (*Juncus* spp.), pickleweed, brass buttons (*Cotula coronopifolia*) andalkali heath (*Frankenia salina*).

The upland area consists of disturbed grasslands that are dominated by nonnative grasses and includes a few areas with fennel (*Foeniculum vulgare*) and coyote brush (*Baccharis douglasii*).

2.1 PROJECT BACKGROUND

2.1.1 Site Regulatory History

The RWQCB is the lead regulatory agency and currently provides oversight of environmental investigation and remediation activities at the Site. The Slough, particularly the northern segment, has been the subject of several environmental investigations to evaluate metals concentrations in soil and sediment. Environmental investigation work was performed by CH2M Hill in 1986, the Bay Protection and Toxic Cleanup Program in 1991, and RWQCB in 1995 and 1997. These investigations focused primarily on sediments in the north Slough and identified copper and zinc as the primary COCs. The elevated concentrations of copper and zinc encountered in the Slough sediment were attributed to historical activities associated with disposal of mining spoils, cinders, and slag by MOCOCO. The precipitation infiltration and/or groundwater that came in contact with this waste generated a leachate rich in copper and zinc. This leachate has historically migrated towards and discharged into the slough, where the copper and zinc then precipitate from solution thus producing the contaminated sediments on the slough bottom.

Based on the results of these studies, the RWQCB Bay Protection Toxic Cleanup Program identified the Slough as one of the "toxic hot spots" within the San Francisco Bay Area (RWQCB, 1997). Subsequently, on August 10, 1999, the RWQCB requested under Section 13267 of the California Water Code that Rhodia develop a remedial action plan (RAP) and periodic status reports for the Site to address the COCs within the Slough. An FS was submitted to the RWQCB in March 2001. An addendum to the Feasibility Study and Conceptual Remedial Action Plan was submitted to the RWQCB in January 2002. The contents and recommendations of these documents are summarized in Sections 2.1.4 and 2.1.5 below.

On August 20, 2001 the RWQCB issued the Order No. 01-094 entitled *Site Cleanup Requirements for Rhodia Inc. Martinez Facility, Peyton Slough Project* (the Order). The purpose of the Order was to "adopt cleanup requirements for sediment contamination in and adjacent to Peyton Slough". The Order is included in Appendix A. The Order outlines a series of tasks and compliance dates for each task. The tasks and compliance dates outlined in the Order are as follows:

- Task 1: Expanded Groundwater Monitoring Plan and Well Installation
  - 1a Monitoring Plan: August 15, 2001
  - 1b Documentation of Monitoring Well Installation: November 15, 2001
  - 1c First Quarterly Report: April 30, 2002
- Task 2: Risk Assessment and Groundwater Evaluation to Establish Groundwater and Soil/Sediment Cleanup Standards: December 15, 2001.
- Task 3: Remedial Design Report and Implementation Schedule: March 15, 2002
- Task 4: Groundwater Cleanup Plan and Schedule to Implement Cleanup Plan: submittal within 120 days from confirmed release.
- Task 5: Documentation of Remediation of Peyton Slough:

- Physical Remediation Work Completion: December 31, 2002
- Documentation Report: April 15, 2003

- Task 6: Proposed Institutional Constraints: May 15, 2003
- Task 7: Implementation of Institutional Constraints: July 15, 2003
- Task 8: Five-Year Status Report: April 15, 2008

Tasks 1a, 1b, 2 and 3 have already been completed. The Monitoring Plan was submitted to the RWQCB on August 15, 2001. The Documentation of Monitoring Well Installation was submitted to the RWQCB on November 9, 2001. The Risk Assessment and Groundwater Evaluation was submitted to the RWQCB on February 15, 2002 (with concurrence of the RWQCB for the modified due date). This report has been prepared in response to Task 3 of the Order and was originally submitted March 15, 2002.

The site is also subject to the following RWQCB requirements: Waste Discharge Requirements Order No. 97-12 (October 15, 1997), and National Pollutant Discharge Elimination System (NPDES) Permit Order No. 98-104, Permit No. CA0006165 dated October 28, 1998.

2.1.2 Summary of Previous Studies (Harding Lawson Associates 1998, 2000)

Environmental investigation work was performed by Harding Lawson Associates (HLA) in 1998 and 2000. The 1998 investigation focused on the sediments at the bottom of the north Slough (between the tide gate and Carquinez Strait). The investigation found elevated levels of copper and zinc in the sediment. The 2000 investigation focused on the sediments at the bottom of the south Slough (between the tide gate and Waterfront Road). Copper and zinc were detected in all samples analyzed at concentrations ranging from 43.5 to 71,700 milligrams per kilogram (mg/kg), and 100 to 88,300 mg/kg, respectively.

2.1.3 Pre-Dredging Site Investigation (URS, 2000)

The Pre-Dredging Site Investigation was completed and submitted to the RWQCB on December 28, 2000. The purpose of the pre-dredging investigation was to identify potential areas of concern (AOCs). For the pre-dredging investigation report, potential AOCs where defined as areas in, or immediately adjacent to, the Slough with concentrations of copper and zinc exceeding Effects Range-Median (ER-M) values or with low pH. The pre-dredging site investigation included the collection of sediment samples from the slough bottom, slough sidewalls, from side tributaries to the north Slough, and from the dredge spoil piles immediately adjacent to the Slough. In addition, nine trenches and three test pits were excavated along the western embankment in the vicinity of the tide gate to investigate whether cinders and surface fill are present in the slough sidewalls, and to determine the presence/absence of the former paleo-channel. The results of this and previous investigations are discussed in Section 3.2 AOC Delineation.

The ER-Ms for copper and zinc were exceeded in the Slough bottom, in limited sidewall samples, and in areas along the embankment where dredge spoil piles are located adjacent to the Slough. Based on the results of this and previous investigations, the Slough and adjacent dredge spoil piles were defined as potential AOCs.

The samples collected from the trenches closest to the Slough contained elevated levels of metals. However, no cinders were found to be present with the exception of a thin layer found under the surface fill in Trench 2. The current slough sidewalls do not appear to be constructed of, or contain, cinders.

Based on the findings of this investigation, URS recommended the following:

- Prepare a feasibility study/remedial action plan (FS/RAP) to evaluate remedial options for the areas identified as potential AOCs.
- Schedule a meeting with permitting agencies to discuss the California Environmental Quality Act (CEQA) and permitting requirements for upcoming remedial activities.

2.1.4 Feasibility Study

The FS was completed and submitted to the RWQCB on March 2, 2001 (URS, 2001a). The FS was developed for the potential AOCs identified in the Pre-Dredging Investigation (URS, 2000a). Seven remedial action alternatives were initially screened based on regulatory acceptance and technical implementability. The initial screening produced four viable remedial action alternatives, which were further evaluated based on seven criteria: protection of human health and environment; compliance with remedial action objectives; short and long-term effectiveness and performance; reductions in toxicity, mobility, and volume through treatment; implementability; cost; and regulatory and community acceptance. Based on this analysis, Alternative 6a (Mechanical Dredging to a Depth of 3 Feet with Silt Screen, Landfill Disposal, Capping, and Institutional Controls) and Alternative 7b (Full Re-alignment of Peyton Slough, Capping and Backfilling of the Existing Slough Alignment, Restoration of Marsh, and Institutional Controls) emerged as the preferred alternatives. Typically the FS process identifies one alternative as the preferred alternative. However, at the time of submittal of the FS, several environmental permitting issues and technical questions remained unresolved for both alternatives. This precluded the selection of one preferred alternative. The final preferred alternative was selected and described in the AFS/RAP (URS, 2002a).

2.1.5 Addendum to FS and Conceptual Remedial Action Plan (AFS/RAP)

Subsequent to the March 2, 2001 submission of the Feasibility Study, on August 20, 2001, the RWQCB issued the Order. In a subsequent letter dated November 5, 2001, the RWQCB requested an addendum to the FS that would indicate which remedial alternative is preferred and providing sufficient evidence of which alternative best meets the project goals as defined in the Order. Since the submittal of the FS Report, Rhodia and URS have met with potential contractors and other regulatory agencies to assess the technical and regulatory issues associated with each of the two alternatives selected in the FS.

The AFS/RAP was completed and submitted to the RWQCB on January 10, 2002 (URS, 2002a). The AFS/RAP concluded that Alternative 7b - Full Re-Alignment of the Peyton Slough was the most feasible, effective, and readily implementable alternative. While both Alternatives 6 and 7b meet the remedial action objectives to varying degrees, the full re-alignment alternative provides for additional positive impacts and opportunities for future benefits, and is the most cost-effective solution. The full re-alignment alternative minimizes the potential for long-term, ongoing risk to sensitive receptors by (1) allowing future maintenance or habitat enhancement dredging without the potential for resuspension and exposure of contaminated sediments, and (2) minimizing the potential migration of contaminants to the new alignment.

The full re-alignment alternative involves the construction of a new alignment from approximately 200 feet north of the Waterfront Road railroad culvert to Carquinez Strait. The full re-alignment will be located to the east of the existing Slough alignment. The existing Slough will be dewatered, and backfilled with an engineered cap designed to isolate the COCs in the sediments from habitat. The engineered cap will provide substrate to restore portions of the existing Slough to wetland habitat.

The RWQCB November 5, 2001 letter also requested an "Interested Party and Permit Status Report." This report was attached to the AFS/RAP and included a summary of the contacts made to date by Rhodia or URS on behalf of Rhodia with the San Francisco Bay Conservation and Development Commission (BCDC), the US Army Corps of Engineers (USACE), the California Department of Fish and Game (Region 3), the California State Lands Commission, Shore Terminals LLC, the CCMVCD/McNabney Marsh Advisory Committee/MVSD, and Kinder Morgan.

2.1.6 Risk Assessment

The Risk Assessment (RA) was submitted to the RWQCB on February 15, 2002. The purpose of the RA was to provide a basis for evaluation and delineation of AOCs, evaluate groundwater conditions adjacent to the existing Slough and the new alignment, and to propose target cleanup levels for soil/sediment that may remain in place after the remedial alternative is implemented. After identifying data gaps in the historical data, URS collected additional samples to characterize the Site in 2001 and 2002, including soil, sediment, surface water, groundwater, plant tissue, and invertebrate tissue samples. The results of this and previous investigations are discussed in Section 3.2 AOC delineation. Toxicity tests based on sediment elutriates and bioaccumulation studies were also performed.

The risk assessment was conducted using a tiered approach that focused the assessment on the areas that warrant remedial attention. In Tier 1 (screening level), Risk-Based Screening Levels (human health) or ER-Ms (ecological), were used to screen representative concentrations in media from the existing and new sloughs. In Tier 2, site-specific target levels (SSTLs) were developed for human and ecological receptors and for exposure scenarios that were appropriate for the Site.

<u>For the wetland habitat.</u> A two-step process was adopted for the use of target cleanup levels for soil and sediment that may remain in place after the remedial action is implemented. In the first step, sediments with residual concentrations of chemicals where the probability of adverse effects is high will be identified for removal or remediation efforts (i.e., concentrations exceed the lower of the ER-Ms or lowest-observable-adverse-effects level [LOAEL]-based SSTLs). In the second step, sediments with residual concentrations where the potential for adverse effects is moderate will be proposed for in-place risk management or for other risk reduction measures that may not necessarily include removal (i.e., concentrations are lower than ER-Ms or LOAEL-based ecological site-specific target levels (E-SSTLs) but exceed no-observable-adverse-effects level [NOAEL]-based E-SSTLs).

The target cleanup goals were selected after review of the SSTLs for all the evaluated wetland receptors. The target cleanup goals that were initially proposed for the first step (effect-based SSTLs) were 270 mg/kg for copper and 410 mg/kg for zinc for wetland habitats, based on the ER-Ms. Exceedance of these values indicates the potential for toxicity to the benthic community. Sediments that exceed these values would be proposed for active remediation. The target values that were initially proposed for the second step were 239 mg/kg for copper and 251 mg/kg for zinc in the wetland habitat (protective of the salt marsh harvest mouse). Sediments that exceed these values but that are below the ER-Ms were recommended for in-place management and/or risk reduction.

A subsequent sampling of the North Peyton Marsh in the vicinity of the proposed new alignment shows surface sediments in the area to contain concentrations of copper and zinc that are substantially greater than expected. Based on this new data, the ambient concentrations of copper and zinc in surface sediments in the North Peyton Marsh is determined to be 349.24 ppm and 1,335.64 ppm, respectively. Use of the initially proposed target cleanup goals might require removal of the entire surface of North Peyton Marsh, an action which is neither necessary, desirable, nor permittable. As a result of this finding, a greater percentage of the AOC will be managed in place.

For the aquatic habitats. Only one set of target cleanup levels was adopted, the ER-Ms (270 mg/kg for copper and 410 mg/kg for zinc). Because the NOAEL-based E-SSTLs for the aquatic habitat were higher than the ER-Ms, use of the NOAEL-based E-SSTLs to identify areas suitable for in-place management may not be protective of the benthic community. Therefore, sediments exceeding the ER-Ms in aquatic habitats are proposed for active remediation.

The results of the Ecological RA indicated that the majority of the dredge spoil piles along the northern and southern portions of the existing Slough exceeded ER-Ms for copper and zinc. In addition, all the samples from the spoil piles that were subjected to toxicity testing proved to be significantly toxic. SSTLs for food-web-based ecological receptors also were exceeded in many areas of the spoil piles. Similarly, sediments in the bed of the existing Slough, and to a lesser extent, in the embankments along the sides, had numerous exceedances of ER-Ms. The human health SSTLs for the higher elevation upland portions of the Site were not exceeded by Site concentrations of copper and zinc. However, sediments in portions of the existing Slough north of the tide gate did exceed the human health SSTLs for fish consumption. Therefore, based on their potential for impacts to human health and the environment, the dredge spoil piles along the sides of the length of the existing Slough are proposed as AOCs. Additionally, sediments and surface water in the existing Slough, where risk-based SSTLs are exceeded are also proposed as AOCs.

The groundwater evaluation assessed the potential for groundwater from the Slough vicinity to impact sediment and surface water quality in the new alignment. The results of this evaluation indicate that, provided the dredge spoil piles are removed from areas south of the tide gate, groundwater impacts to the new alignment (both north and south of the levee) will not be significant. Risks through exposure to surface water will be minimized when the existing Slough is rerouted into the new alignment.

2.2 ASSESSMENT OF CONTAMINATION SOURCES TO THE AOCs

The main sources of contamination (namely copper and zinc) to the AOCs were waste by-products (cinders and slag) from a copper ore smelting operation that took place on Rhodia's property during the first half of the 20th century. MOCOCO operated a copper ore smelter from approximately 1899 to 1923 and a pyrite roaster until approximately 1966. During the copper smelting and roasting operations, a large pile of cinders and slag was deposited on the property. When the copper smelting operation ceased, portions of the adjacent marsh and low-lying portions of the property were filled to grade with copper smelter slag and cinders taken from a nearby cinder/slag pile and then covered with imported fill material. A portion of the cinder/slag pile on Rhodia's property that subsided below grade was also covered in place, including portions of the paleo-channel on the Property.

Groundwater flow is believed to be the primary mechanism by which contaminants are conveyed to the Slough. Observations made at the site indicate that the cinder filled paleo-channel may be a groundwater conduit from the ore bodies to the existing Slough, and that contaminated groundwater may be seeping into the existing Slough immediately south of a tide gate and levee located on Rhodia's property. Shallow groundwater samples collected from this area contain high levels of copper and zinc (URS 2002). The subgrade portions of the old cinder/slag piles also contribute contaminated groundwater that flows through a shallow near- surface zone made up of cinders/slag and fill. Groundwater in the shallow zone is believed to flow into the Slough south of the tide gate. During periods of high rainfall, when groundwater levels are at their highest, this shallow groundwater flow surfaces and enters the Slough as overland flow south of the tide gate. Rainfall falling on unpaved portions of the site contributes to the shallow groundwater flow as well as to overland flow into the Slough. Dissolved concentrations of copper and zinc have, therefore, been slowly contaminating the adjacent Slough via groundwater transport.

Another source of contamination was created on the project site by the routine historical dredging of the Slough. Dredge spoil piles containing COCs were deposited along the banks of the Slough, which in turn exposed the adjacent marshes, North and South Peyton Marshes and Rhodia Marsh. Many of the dredge spoil piles have eroded, particularly in the area south of the levee that runs east-west through the project area (see Figure 1.2-3). The Slough was last dredged in the early 1980s.

2.3 PROJECT SITE

The project site is the area where the construction activities are going to take place. The majority of the project site is located within the reaches of the existing Slough between Zinc Hill, Waterfront Road, the Property and Carquinez Strait (see Figure 1.2-3). The new alignment will run parallel to and to the east of the existing Slough, breaching the levee that separates the northern and southern sections of the project site. The relocation of the south alignment may require the purchase of land or access rights from the adjacent property owner. The relocation of the north alignment may require a land lease from the California State Lands Commission (see Figure 1.2-2). The project site also contains some upland areas in and around the Slough including a levee, the Property, and an access road along the base of Zinc Hill.

2.4 REMEDIAL ACTION OBJECTIVES

The present conditions in the Slough and adjacent spoil piles are not suitable for a healthy, viable benthic community or, indirectly, for higher trophic level species. The ultimate objective of the proposed remediation project is to restore sediment quality and, in turn, create a safe and suitable environment for humans and wildlife. There are several specific objectives that must be met by the design and the implementation of the remedial actions in the AOCs. These objectives have been set based on regulatory orders, policies, plans, and good engineering practice, and are listed below:

- Compliance with the requirements of the Order
- Restoration and protection of the beneficial uses as identified in the 1995 San Francisco Water Quality Control Plan (Basin Plan) for Peyton Slough as a tributary to Carquinez Strait, including:
- Fish spawning
- Wildlife habitat
- Fish migration
- Preservation of rare and endangered species
- Estuarine habitat
- Ocean, commercial, and sport fishing
- Industrial service supply.
- Protection of human health and the environment by elimination of ongoing risk to sensitive receptors
- Long-term physical and chemical isolation of the deeper sediments containing COCs via the installation of an engineered cap

2.5 PROJECT APPROACH AND PROPOSED REMEDIAL ACTION

The project approach developed for this remediation project addresses the site-specific conditions, and integrates the remedial action objectives listed in Section 2.4 with the engineering analysis of the remedial action described in Section 3. The project approach is based in the following two major elements:

- The focus of the project is the “toxic hot spot” identified by the RWQCB Bay Protection Toxic Cleanup Program. The remedial action focuses on the COCs in sediment at the bottom and sidewalls of the Slough and in the dredge spoil piles along the embankment.
- The remedial action must comply with the remedial action objectives, and specifically show through an engineering analysis that the following design criteria are met:
- Isolation of the COCs in sediment in the bottom of the Slough
- No ongoing risk to receptors

- Protection of new alignment
- Improved habitat
- Functioning hydraulic system
- Long-term effectiveness

A number of alternatives were evaluated as part of the FS (URS 2001) and the subsequent AFS/RAP (URS 2002a) processes. This RDR is based on the selection of Alternative 7b-Full Re-Alignment of the Slough as the preferred alternative. The full re-alignment alternative includes land-based and barge-based excavation of a new alignment, removal of dredge spoil piles, backfilling/capping of existing Slough, and creation and enhancement of wetlands. The proposed project plan view is shown on Figure 2.5-1. Compared to other alternatives, the full re-alignment alternative provides for the solution that best meets the remedial action objectives, provides additional positive impacts and opportunities for future benefits, and is the most cost-effective solution.

The full re-alignment alternative creates an uncontaminated slough substrate that allows future maintenance or habitat enhancement dredging and at the same time minimizes the potential risk for resuspension and exposure of the existing Slough to contaminated sediments. The capping and backfilling of the existing Slough provides isolation of COCs from habitat and obstructs the exposure pathway for sensitive receptors. The capping and the re-alignment of the existing Slough will also minimize the potential for migration of COCs to the new alignment. As a long-term benefit from the fill, the placement of the engineered cap in the existing Slough will "minimize harmful effects to the Bay Area, such as, the reduction or impairment of the volume of surface area or circulation of water, water quality, fertility of marshes or fish or wildlife resources, or other conditions impacting the environment..." (as stated in and required by the McAteer-Petris Act, Section 66605[d]). By preventing further disturbance of contaminated sediments and by chemically and physically isolating those sediments from the habitat via capping the existing Slough, the re-alignment alternative is also an effective long-term remedial action for the Site.

The loss of existing Slough habitat, as well as habitat in the marsh land along the new alignment, will be mitigated by the restoration of the marsh on the north and south ends of the Slough, by the creation and enhancement of wetlands in the Peyton Marsh system, and by the creation of a cleaner and more productive slough habitat. The project site plan view upon the completion of the remedial activities is shown on Figure 2.5-2.

3.1 OVERVIEW OF REMEDIAL ACTION

This RDR is based on the selection of Alternative 7b - Full Re-Alignment of the Slough as the preferred alternative for remedial action. The full re-alignment alternative proposes a new alignment to begin approximately 200 feet from the railroad crossing at Waterfront Road and continue to Carquinez Strait (connecting to the existing small slough, Peyton Slough No. 1, located approximately 600 feet east of the Slough). The remedial action includes land-based and/or barge-based excavation of the new alignment, removal of AOCs, backfilling/capping of the existing Slough, relocation of the tide gates into the new alignment, and restoration and creation of wetlands (see Figures 2.5-1 and 2.5-2).

The new alignment will run parallel to and east of the existing Slough and will be designed with a hydraulic capacity equivalent to or greater than that of the existing Slough. The backfilling/capping of the existing Slough will require the placement of an engineered cap to isolate the COCs and convert the majority of the existing Slough alignment into habitat. The cap elevation will be determined based on the type of habitat to be created and actual field conditions.

Section 3 includes the data evaluation and design to support the selected alternative. Specifically, Section 3 describes the engineering analysis performed to complete the design of the remedial action. The elements of engineering analysis necessary for this alternative are as follows:

- Section 3.2 discusses delineation of the AOCs and includes results from historical sampling and additional data collected as part of this RDR for further AOC delineation.
- Section 3.3 includes the analysis of hydraulic conditions to provide the depth, width, bottom elevation and slope of the new alignment.
- Section 3.4 includes the evaluation of hydrogeologic conditions at the project site to provide a better understanding of the potential pathways of contamination to the new alignment. Section 3.4 also includes the preliminary tidal study.
- Section 3.5 includes the analysis of contaminant transport mechanisms that could impact the new alignment.
- Section 3.6 presents the results of the investigation, analysis, and modeling conducted to design an engineering cap that effectively isolates the COCs in the existing Slough alignment.
- Section 3.7 includes the geotechnical evaluation of tide gate relocation and a preliminary design of the tide gate structure.
- Section 3.8 includes the review of existing site drainage conditions, and conceptual design of engineering controls on the Rhodia Property adjacent to the Slough.

3.2 AOC DELINEATION

This section provides a description of the RDR delineation transect sampling activities, an evaluation of the analytical data, and a delineation of the AOCs based upon both historical and

RDR chemical sampling results. This section also includes a delineation of the AOC and an evaluation of AOC removal alternatives based on current site conditions.

This section has the following subsections:

- Field Sampling Activities
- Evaluation of Results
- Conceptual Plan for AOC Removal and Management in Place
- Conclusions

Delineation of AOCs at the site was focused mainly on the dredge spoil piles and fringes of the piles along the east and west side of the existing Slough. These piles, and the sediments in the bottom and sidewalls of the existing Slough, were identified in the Order as potential remedial areas needing evaluation. The AOC extent was initially estimated by comparing historical and new data with the ER-Ms and the upper confidence limit (UCL) for copper and/or zinc. The denuded tops of the dredge spoil piles were identified as AOC based on data collected from previous investigations. Over time, some of the piles have been eroded during periods of inundation resulting in the spread of COCs away from the Slough banks. The purpose of the additional delineation effort conducted under this RDR, and subsequently during the preparation of the Initial Study (URS 2002) was to identify the extent of the COCs and delineate the AOC for removal, capping, or management in place.

By design, the existing Slough and sidewalls will be capped, as discussed in Section 3.6. The dredge spoil piles and resulting spread of pile material will be evaluated based on the following criteria:

- Elevated Ambient Concentration
- Reduce Impacts to Wetland Habitat

While the AOC was initially delineated by the numerical criteria and depositional features, its removal or management in place as a remedial action will be evaluated further based on the above criteria. Where elevated ambient concentrations are present, AOC removal may not be warranted, recognizing the presence of ambient copper and zinc concentrations in sediment in the North Peyton Marsh caused by other historical uses outside of the Peyton Slough contamination issues. Furthermore, the removal of AOC, where high quality pickleweed stands (indicative of healthy wetlands) are prevalent, may not be the most appropriate approach. Management in place of these AOCs would serve to preserve the high quality habitat, reduce impacts caused by temporal losses to (high quality) wetland habitat, and allow for completion of the remedial action within a two-year period, further reducing temporal impacts to the wetlands.

Figures 3.2-1 through 3.2-4 present historical sampling results for copper and zinc in the existing slough bottom and embankments, and surface and subsurface soil/sediment adjacent to the existing Slough and along the new alignment. Figure 3.2-5 shows locations of the delineation transects (DTs) sampled in this additional investigation for the RDR and the analytical results for copper and zinc at those locations.

3.2.1 RDR Field Sampling Activities

This subsection describes the field sampling activities related to soil and sediment sampling in the AOCs targeted for removal or management in place. Sampling activities were conducted throughout the preparation of this RDR and during the preparation of the Initial Study (URS 2002).

Figures 3.2-1 through 3.2-4 present historical sampling results for copper and zinc in the existing Slough bottom, existing Slough embankments, and surface and subsurface soil/sediment adjacent to the existing Slough and along the new alignment. Figure 3.2-5 shows locations of the delineation transects (DTs) sampled in this additional investigation for the RDR and the analytical results for copper and zinc at those locations.

Delineation transects (DTs) were laid out to delineate the lateral extent of the dredge spoil pile fringes to the northeast, northwest, and southwest of the existing Slough. The DT locations were selected based upon previous sampling points identified as having copper and/or zinc concentrations above ER-Ms. Transects were laid out perpendicularly to the Slough and lateral sample intervals were marked at the six intervals starting from the edge of the pile/vegetation line. Figure 3.2-6 is a generalized cross section of a DT showing depths and locations of the sample intervals.

Additional sampling in North Peyton Marsh was conducted in June and July 2002. Figure 3.2-7 illustrates the sample locations.

Delineation Transects in Dredge Spoil Piles. At each of the nine DTs, the sampling strategy called for collection of soil samples from six locations, three east of the dredge spoil pile and three west of the dredge spoil pile spaced 15, 30, and 60-feet away from the edge of the dredge spoil pile. Three samples were collected at each of the 6 locations at depth intervals of 0 to 6, 6 to 12, and 12 to 24 inches below ground surface (bgs). Transects were numbered DT-1 through DT-9 and sample locations were labeled as follows:

- 1E/1W for sample locations approximately 15 feet from vegetation line in direction indicated (east or west);
- 2E/2W for sample locations approximately 30 feet from vegetation line in direction indicated;
- 3E/3W for sample locations approximately 60 feet from vegetation line in direction indicated; and
- In DT-6, an additional sample at 4W was collected approximately 75 feet from the vegetation line.

Sample identifications used on laboratory chain of custody forms and referred to in this report are set up as follows:

- **DT - #** *(1-9)* – **Location** *(1,2,3)* **Direction** *(E,W)* – **Depth** *(0-6,6-12, 12-24)*

Therefore, the sample identification (ID) for the DT-1 sample 15 feet east of the vegetation line from 0 to 6 inches is **DT-1-1E-0-6**.

Soil samples were also collected from six locations along the existing Slough sidewall (embankment samples) at the same three depth intervals (Embankment 1 through Embankment

6). Figure 3.2-5 shows the approximate locations of the embankment samples and the analytical results for copper and zinc.

Soil samples were collected from the desired depth by hand pushing a sample tube or by hand auguring. Hand augured soil samples were placed in laboratory-prepared jars. The desired depth interval of the hand pushed sample tube was sealed and capped. Samples were labeled with the sample ID and interval, date and time of sampling, and contact information. Samples were retained in laboratory-prepared containers, and transported under chain-of-custody procedures to the laboratory, where they were homogenized within the sample interval and analyzed for copper, zinc, pH and moisture content. For quality assurance/quality control (QA/QC) purposes, ten percent of the soil samples were collected for duplicate analysis.

All samples in the delineation transects were placed on hold at the laboratory and analyzed in a phased approach. The most shallow and closest samples to the piles were analyzed for copper, zinc, moisture content and pH. Additional phases of analyses followed according to each set of results received. These delineation transect samples pre-dated the North Marsh background sampling. As a preliminary screening, the ER-Ms were used to guide the analytical effort. Where copper or zinc concentrations were found in exceedance of ER-Ms, the next deeper depth interval and next lateral interval away from the pile were analyzed for the exceeding analyte. In most cases, when a sample was below the ER-M for both analytes, no further analyses was conducted.

The AOC is discussed based on the physical location of the dredge spoil piles, and includes the following three areas:

- Dredge Spoil Piles North of Levee – The area north of levee to the east and west of the existing Slough (Note: There is only one pile on the west side of the Slough.)
- Dredge Spoil Piles South of Levee – The area south of the levee to the west of the existing Slough
- South Spread Area and Southeast Dredge Spoil Piles – The approximately 9.62-ac area south of the levee between the existing Slough and the toe of Zinc Hill, which includes approximately 1.76 acres of dredge spoil piles south of the levee and east of the existing Slough

North Peyton Marsh. Sampling activities were undertaken in the North Peyton Marsh between July 25 and July 29, 2002 by URS. The samples were laid out on a 100- by 400-foot grid with a centerline that followed the new alignment. The grid had six east-west transects with five sample locations on each transect, at approximately 100-foot centers. The latitude and longitude of all sample locations was recorded using a high-accuracy GPS unit.

Sediment samples were collected from two depth intervals at all 30 locations. A hand auger was used to collect a soil sample within the first 1.5 feet bgs, representing the root-mat zone. Below the root-mat zone, at approximately 2 feet bgs, the matrix turned to a high moisture content slurry. In this matrix, a discreet sludge sampler was used to collect a sample at approximately 6 feet bgs. For each sample, an 8 oz. soil sample jar was filled, labeled, and sealed. Samples were then placed in a chilled ice chest and sent to Chroma Laboratory under chain-of-custody procedures. For quality assurance/quality control (QA/QC) purposes, ten percent of the soil samples were collected for duplicate analysis. The samples were analyzed for copper, zinc, pH, TOC, and moisture content.

3.2.2 Evaluation of Results

Table 3.2-1 presents the analytical results for copper and zinc compared to ER-Ms for the DT samples and embankment samples. ERMs were chosen for comparison in order to develop a conservative estimate of the amount of AOC to be removed. Subsequent sampling in North Peyton Marsh has established that the ambient background concentrations of copper and zinc are substantially higher than the ER-Ms. The limit of the AOC will be recalculated using the ambient background concentrations of copper and zinc as a basis for determining the boundary of the AOC for removal.

Sample locations and chemical data are presented on Figure 3.2-5. Copies of laboratory analytical reports are included in Appendix B-1.

Northeast Dredge Spoil Piles. The dredge spoil piles in the northeast area range in elevation from +3.2 feet NGVD to +7 feet NGVD, and have some vegetation (Coyote Bush and upland grasses). Although not all denuded areas exhibit copper and zinc concentrations exceeding ER-Ms, evidence of denuded drifts near the edges of the piles suggests that some of the piles near DT-1 have spread up to approximately 60 feet east from the transect center and as far west as the existing Slough. The piles at DT-3, DT-4 and DT-5 are vegetated, as are the surrounding wetlands. Because these piles are located within a close proximity of the CCMVCD drainage ditches, there is less tidal influence which has apparently resulted in less spreading of the dredge spoil material.

The northeast piles are discussed in two parts: the piles in the northern portion DT-1 and DT-3, and the southern piles DT-4 and DT-5. The results from northern transect samples indicate that copper concentrations diminish to below the ER-Ms as distance from the center of the piles increases. Copper was found at concentrations below the ER-M (270 for copper) within 15 feet to the east and west within the pile. Zinc concentrations remain relatively stable irrespective of distance from the piles, ranging from just over the ER-M (410 for zinc) at 500 mg/kg up to 2,000 mg/kg. Concentrations exceeding the ER-M for zinc were identified up to 60 feet from the pile and up to 2 feet deep in the samples collected from the location 15 feet from the pile. In historical samples, however, the concentrations of zinc diminish to approximately 100 to 150 mg/kg within 100 feet east of the piles, indicating that the estimated horizontal limit of zinc and copper ranges from 60 to 100 feet from the piles in the northern section of the northeast dredge spoil pile area.

In DT-4 transect samples, the extent of copper and zinc concentrations above the ER-Ms is 30 feet to the east of the pile and extends to the Slough on the west of the pile. These exceedances range in depth from 12 to 24 inches bgs. Copper ranges from 440 to 1,500 mg/kg, and zinc concentrations range from 530 to 1,100 mg/kg.

In DT-5 transect samples, copper and zinc concentrations were encountered above the ER-Ms (up to 1,400 mg/kg copper and 1,900 mg/kg zinc) to 60 feet east of the piles and extending to the Slough on the west of the pile. These exceedances range in depth from 6 to 24 inches bgs.

There is one dredge spoil pile on the northwest side of the existing Slough (around which the DT-2 transect was located). This pile is relatively flat and marked by mostly denuded soil. The highest concentrations in all transect samples were encountered in this location. Copper ranged from 320 to 2,200 mg/kg, and zinc ranged from 880 to 6,300 mg/kg up to 60 feet west from this pile. Concentrations did not diminish with depth up to 2 feet bgs.

Southwest Dredge Spoil Piles. In the area south of the levee and west of the existing Slough (DT-6, -7, -8 and -9), copper and zinc concentrations are generally lower, diminishing to below ER-Ms within the piles, compared to the northeast and northwest areas. The concentrations of copper and zinc diminish rapidly both laterally and vertically from the tops of the denuded areas, likely because the piles are less eroded in the southwest area compared to the northeast area where the edges of the piles are subjected to the daily tide. The COCs in the southwest area appear to fall within a nominal 30-foot radius from the denuded areas, except in the pile at DT-7, where all concentrations are below the ER-Ms.

South Spread Area and Southeast Dredge Spoil Piles. The south spread area and southeast dredge spoil pile area is characterized by low-lying land, mainly denuded, between the existing Slough and the toe of Zinc Hill. The conditions in the south spread area improve in the southern half, where only dredge spoil piles are denuded and there is dense vegetation from the piles up to the new alignment. It appears that the area of spread and denuded piles is limited by the levee to the north and the oxbow formed from the former paleo-channel to the south.

Previous sampling indicates elevated concentrations of copper and zinc in the south spread area up to 2 feet below the general marsh plain elevation of -0.5 to +0.5 feet NGVD. Copper concentrations range from 445 to 3,300 mg/kg in the surface soils. In the samples just below the surface in the 0.5 to 1 foot below marsh plain, concentrations range from 31 to 560 mg/kg. Few samples in the south spread area at 1 to 1.5 foot depth interval had concentrations of copper above the ER-Ms. Zinc concentrations ranged from 198 to 643 mg/kg in surface samples. Samples with elevated concentrations of copper and zinc at the surface were encountered in the oxbow (SSB-11), and in the borrow pit (SSB4).

The dredge spoil piles in the south spread area have historically exhibited elevated concentrations of copper and zinc throughout depth of the piles to grade. In the south spread area adjacent to the piles, historical sampling results show copper concentrations range from 820 to 2,500 mg/kg at the surface, and diminish to between 680 and 1,200 mg/kg with depth. Zinc concentrations range from 610 to 820 mg/kg at the surface, and diminish to between 336 and 680 mg/kg with depth.

Recent and historical sampling events conducted in dredge spoil pile and spread areas have shown that the general contaminant trend demonstrates diminishing concentrations of copper and zinc below the upper 1 to 2 feet of sediments. According to this most recent delineation investigation, the lateral extent of copper and zinc appears to be larger in the northern areas of the south spread area and within a nominal 60-foot radius around the piles.

North Peyton Marsh. Recognizing that the North Peyton Marsh has not been extensively sampled, the RWQCB requested that a series of soil samples be collected in Portion of North Peyton Marsh where the new alignment is proposed to be constructed. The purpose of this sampling was to provide the RWQCB with the data necessary to show that the selected alignment would be located in an environment that would not result in a contaminated slough in the future.

The analytical results are presented in Table 3.2-2. Copper and zinc concentrations in the surface samples range from 61 mg/kg to 970 mg/kg and 140 mg/kg to 7,200 mg/kg respectively. The ranges for copper and zinc in the samples collected from a depth of 6 ft bgs are 39 mg/kg to 130 mg/kg and 50 mg/kg to 1,300 mg/kg respectively. The calculated mean concentrations for copper and zinc in the surface sediment samples are 269.90 mg/kg and 856.67 mg/kg respectively. Whereas the calculated mean concentrations for copper and zinc in the sediment samples collected a 6 ft bgs are 65.37 mg/kg and 169.27 mg/kg respectively.

Based on these data, the surface sediments in North Peyton Marsh appear to have concentrations of copper and zinc that are substantially greater than expected. URS has calculated an ambient concentration for copper and zinc for surface sediments in the North Peyton Marsh based on the 95% Upper Confidence Limit of the Mean (UCLM) for each. The calculated ambient concentrations of copper and zinc in the surface sediments of North Peyton Marsh are 349.24 mg/kg and 1,335.64 mg/kg respectively.

3.2.3 Conceptual AOC Removal Plan

The removal of dredge spoil piles will be guided by the project's overall goals to minimize the potential for ongoing risk to sensitive receptors and the potential for recontamination of cap materials or the new alignment. Section 3.5 of this report provides a detailed analysis of the potential for residual concentrations of COCs to impact the capped or restored areas of the project site and the new alignment.

Based on the physical and chemical characteristics of the dredge spoil pile areas, the AOC will be removed and the disturbed areas will be restored to marsh habitat.

Specifically, the maximum lateral extent of removal is defined by meeting the following criteria:

- Removal of the piles up to and including the zones of diminished copper and zinc concentrations to near or below the calculated ambient copper and zinc concentrations for North Peyton Marsh,
- Reaching the marsh plain elevation (approximately 3.2 feet NGVD to the north of the levee, and +0 foot NGVD to the south of the levee) in order to restore the marsh plain to its proper elevation, and
- Removal of localized denuded spread areas.

The criteria listed above are specifically geared to address the removal of dredge spoil pile material as an active mitigation of a source of copper and zinc. Furthermore, this removal action will serve to reduce the potential for ongoing migration of residual concentrations of copper and zinc contaminated materials by creating an area that is level with the marsh plain elevation, and by restoring the disturbed area to marsh habitat.

Following these criteria, the maximum extent of COCs to be removed including piles and spread areas, as shown in Figure 3.2-5, comprises an area of up to 20 ac. The northeast and northwest pile areas may require the removal of pile material to a radius of 60 feet from the vegetation line, or up to the Slough, whichever is closer to the pile. In addition, denuded areas along the fringes of the piles will be scraped to meet the marsh plain elevation. Because the southwest piles were relatively limited in extent, removal of AOC material may be required up to approximately 30 feet east and west from the vegetation line, or up to the existing Slough, whichever is closer to the pile. These disturbed spread areas will also be regraded to meet surrounding cap and marsh elevations, and to restore the excavation areas to marsh habitat, where appropriate.

Under this maximum removal option, all denuded piles would be removed along with a nominal 6-inch layer of material from below the piles. In addition to the piles, areas identified on Figure 3.2-5 as AOCs may also be removed as described below. These disturbed areas will be regraded to meet surrounding cap and marsh elevations, and to restore the excavation areas to marsh habitat see Section 4.4, Site Restoration Plan). (Note: The northern-most pile in the southwest area will not be restored to wetland habitat, because it is an upland area).

However, the extent of AOC removal may be modified based on current site conditions and permitting requirements. The creation of habitat in the south spread area requires the addition of backfill material to rise the surface soil to an elevation suitable for habitat growth. Depending on the type of habitat agreed upon with the RWQCB, the southeast dredge spoil piles and south spread area may be managed in place by capping and/or backfilling the area to the elevation required for habitat creation. The backfill material will provide a planting layer, will rise the surface soil to an elevation suitable for habitat creation, and will be graded into the surrounding marsh and cap. Similarly, other areas within the AOC with existing viable wetlands may also be managed in place to avoid unnecessary impact to wetlands, as described previously in Section 3.2.

Section 3.5 presents the evaluation of potential for contamination in the new alignment using the MIKE model with different scenarios. A scenario was modeled for contamination of the new alignment if dredge spoil piles were left in place in both east and west areas north of the levee. The model showed no risk of potential contamination to the new alignment with the piles left in place.

3.2.4 Conclusions

The maximum extent of AOC material to be removed as part of this remedial design, shown on Figure 3.2-5, comprises and area of approximately 20 ac. This AOC area was determined based on the physical and chemical characteristics of the area using ER-Ms for delineation.

Although full delineation to below ER-Ms for copper and zinc was not reached in some locations up to 60 feet from the piles in the transects north of the levee, there are historical samples which have identified delineation of copper and zinc at locations adjacent to (slightly north or south of) these samples and as close as 100 feet from the piles. In addition, viable wetlands are present in these areas and removal of these viable wetlands would cause additional adverse impacts to habitat that would not be justified. The purpose of the removal action is to mitigate the presence of dredge spoil piles as a source of COCs and to allow for the creation of wetland habitat. Because vegetation in these areas is established, the small volume of residual COCs in the surface soils do not present potential contamination to the new alignment and these areas can be managed in place.

Therefore, due to their limited volume, the proposed residual COC concentrations (primarily zinc) are not expected to represent conditions contrary to the Basin Plan's objectives for protection of beneficial water uses, human health, and the environment. Furthermore, the presence of viable wetlands in this area could be more beneficial than wetlands which may be created in their place following removal of the underlying AOCs.

Similar reasoning could be applied upon agreement with the RWQCB to other areas currently defined within the AOC. This could reduce the area for removal significantly, complying with the objectives for protection of beneficial water uses, human health, and the environment and minimizing the impacts to wetlands.

3.2.5 QA/QC of Analytical Data

Analytical laboratory results were evaluated to assess the quality of individual sample results and overall method performance. Analytical performance was evaluated on both an individual sample and a quality control batch (groups of samples prepared and analyzed together) basis. The data evaluation performed included review of:

- Blanks (method and trip blanks);
- Spikes (laboratory control, and matrix spikes); and
- Sample Integrity (chain-of-custody documentation, sample preservation, and holding time compliance).

The accuracy and precision of the data were found to be acceptable for use of these data in project decisions without qualification. The detailed QA/QC evaluation is included in Appendix B-2.

3.3 CHANNEL DESIGN

3.3.1 Introduction and Approach

This section of the RDR presents the design of the new alignment including: channel width (top and bottom), cross-sectional area, depth, and sidewall slope. The design objective considered for the new alignment is to provide a functioning slough that has a flow capacity equivalent to or greater than that of the existing Slough. The new alignment must discharge upstream flows and be capable of providing muted tidal action to the Peyton Slough marsh system, (North Peyton Marsh, South Peyton Marsh, Rhodia Marsh, and if possible the McNabney Marsh - see Figure 1.2-1). In addition, the new alignment design must consider the following:

- Limiting the sediment buildup by designing for sufficient velocity to scour sediment
- Minimizing permanent loss of wetlands caused by dredging/excavation of new alignment
- Passing storm event flows without causing flooding in excess of existing conditions

The cross-sectional area of the existing Slough was averaged north and south of the tide gate. Calculations indicated that north of the tide gate, the existing channel has an average cross-section of about 160 ft^2 at the marsh plain elevation. South of the tide gate, the existing channel is less clearly defined in some places and has an average cross-sectional area of only about 100 ft^2 or less. As part of the proposed mitigation plan (see Section 4.4 Wetland Mitigation), the hydraulic capacity of the new alignment north of the tide gate will be increased by 20%. This results in an increased cross-sectional area of 200 ft^2 (versus the current average of 160 ft^2), and a top width of 43 feet (versus the current equivalent-average of 38 feet). Exhibit 3.3-1 below summarizes the channel sizes required to achieve these cross-sectional areas south and north of the levee for a channel with a bottom elevation similar to existing Slough (-3.5 feet NGVD). For the calculation of the top and bottom widths, the design slope of the sidewalls was selected so that it would provide embankment stability and minimize existing wetland loss. A slope of 2 to 1 was selected for the entire channel except the approximately northernmost 200 feet, where the soils are sandier and the sidewalls will be constructed at a 4 to 1 slope. The proposed location of the new alignment and representative cross-sections are shown on Figure 3.3-1.

Exhibit 3.3-1
Channel Sizes
(Assume bottom elevation is -3.5 feet NGVD)

Location and side slope	Bottom Width (feet)	Top Width (feet)
North of Levee (top of bank +3 feet NGVD, 2:1 slope, 200 ft^2 cross section)	17	43
North of Levee (top of bank +3 feet NGVD, 4:1 side slope, 200 ft^2 cross section)	5	57
South of Levee (top of bank +1 feet NGVD, 2:1 slope, 100 ft^2 cross-section)	13	31
South of Levee (top of bank +0 feet NGVD, 2:1 slope, 100 ft^2 cross-section)	22	36

This section of the report contains three subsections including this introduction. Section 3.3.2 describes the history and current behavior of the Peyton Marsh system. Section 3.3.3 evaluates the functioning of the Peyton Marsh system with the new alignment. Lastly, Section 3.3.4 provides conclusions and limitations. Elevations in this report are relative to the NGVD 29 datum.

3.3.2 Existing Conditions

The Peyton Marsh system is comprised of the North Peyton Marsh, the Rhodia Marsh, the South Peyton Marsh, and McNabney Marsh. Exhibit 3.3-2 (below) lists the approximate areas of these marshes grouped by location.

Exhibit 3.3-2
Peyton Slough Marsh System Areas

Description	Acres
North of Levee (North Peyton Marsh)	45
South of Levee (south Peyton and Rhodia Marshes)	58
McNabney Marsh	123
Total Area	226

Hydraulic controls and obstructions have been part of the Peyton Marsh system for over 100 years and have dramatically altered the marsh quality and function. This section provides a brief history of activities in the Peyton Marsh system and is included to provide background on how the Peyton Marsh system became the way it is. Much of the information on the history of the Peyton Marsh system comes from *Inventory and Evaluation: Peyton Marsh Drainage System,* *Contra Costa California* prepared for the CCMVCD by JRP Historical Consulting Services (JRP 1997).

During the course explain, in part, the ambient level of metals (notably zinc and copper) in the North Marsh. The significance of these findings is unclear at this point given the visible evidence that the marsh appears to be a viable wetland supportive of important wetland habitats. Indeed, the alignment of the new slough has been adjusted through the development of the Initial Study to minimize the impacts to high quality pickleweed stands. Removal of historic contamination beyond what is required to relocate the slough would destroy the very functionality the RWQCB seeks to preserve. However, the RWQCB and Rhodia agree, that it would be prudent to monitor the surface water and sediments of the new slough alignment for a 10-year period, at frequencies and scope consistent with the proposed monitoring described in Section D.5 (URS 2002g). RWQCB believes the monitoring program would serve two purposes. First, sediment and water quality monitoring provides one additional potential performance indicator for Rhodia's design detailed in the Remedial Design Report, Revision 1 (URS 2002e), which includes removal, capping, and management in place elements. Secondly, additional sediment and water quality data would provide insight into additional possible impacts on slough water quality including discharges from the publicly owned treatment works (POTW), Interstate 680, Waterfront Road, the railroad, and neighboring properties, as well as the slough's interaction with Carquinez Strait. During the course of the Initial Study, RWQCB had learned that the North Marsh may be less than 150 years old. Its formation took place during a period when mining related activities constituted the majority of industrial operations in the area. The RWQCB believes this helps to explain, in part, the ambient level of metals (notably copper and zinc) in the North Peyton Marsh.

It is not known exactly when the first levee was constructed between Zinc Hill and the Property. Mostly likely it was constructed near the end of the 19th century by either the railroad or MOCOCO. When originally constructed the levee did not contain culverts, and therefore, the levee functioned to effectively isolate much of the marsh system from the Bay. When the CCMVCD was created in 1926 one of its first projects was to modify the Peyton Marsh system to promote better drainage. Starting in 1927 the levee was re-built and canals were constructed to increase drainage and reduce ponding of water south of the levee. Also to facilitate drainage from the marsh into the Bay, the CCMVCD installed two 60-inch diameter culverts with tide gates (allowing drainage from the area south of the levee, and impeding tidal exchange to the south of the levee) to reconnect the marsh system back to the Bay.

The CCMVCD conducted regular maintenance of the marsh system with frequent dredging of the channels and several improvements to the levee over the next 40 years all to facilitate the drainage of the marsh. By the 1940s freshwater inflows to the marsh system from the MVSD and Pacific Gas and Electric (PG&E) had started to cause tule growth. The channels south of the railroad were deepened to increase flow capacity and to prevent the growth of tules, which had started to block the drainage canal. By the 1960s, the flows in the channel had become too large for the tide gates alone to drain McNabney Marsh and a 3,000-gpm pump was added to attempt to abate this condition.

In April 1988, there was a 440,000-gallon oil spill, which occurred and was contained in McNabney (formerly Shell) Marsh, covering the surface of the marsh with oil. According to the McNabney Marsh Advisory Committee (MMAC), the spill caused many areas of the wetland to become "barren of vascular plants or invaded by exotic, weedy, or less desirable plant species"

(MMAC, 2000). The McNabney Marsh Natural Resources Management & Monitoring Plan (October 1988) was developed and included application of chemical herbicides to remove tules and cattail from the Marsh.

In 1998, the MMAC commenced its action to alleviate poor drainage of fresh water from McNabney Marsh to eliminate the bypass and pumping, and to introduce tidal exchange. It was believed that the introduction of tidal exchange would help to eliminate the practice of herbicide application. Accordingly, the existing 60-inch diameter gated culverts were replaced with the existing tide gate structure and additional work was done on the levee. The existing tide gate structure consists of five 4.5 ft x 6 ft box culverts with tide gates: three flap gates and two Nekton gates. The two Nekton gates are similar to flap gates but provide the primary daily drainage. The three flap gates are primarily for draining storm flows from the marshes, and can be manually opened to allow Bay water to enter the marsh system south of the levee. The Nekton gates are now adjusted such that only one gate provides daily drainage. The other gate only opens during large flow events (personal communication with Karl Malamud-Roam, CCMVCD 2002).

Since removal of the two 60-inch diameter gated culverts and other changes to the drainage system in 1998, McNabney Marsh, South Peyton Marsh and the Rhodia Marsh have retained water and have not drained. Even during the summer, there is a permanent pool of water in McNabney Marsh. Historically, McNabney Marsh had dried out in the summer. An initial analysis of the capacity of the five culverts indicated that they have sufficient capacity to drain the three Marshes, if the tide gate were fully functional. However, because subsidence has lowered the elevation of the wetlands to sea level or lower, (e.g., much of McNabney Marsh is 1 foot or more below sea level), it is difficult to drain the marshes. (MLW is about at elevation –1 foot NGVD and MLLW is about –1.7 feet NGVD at Port Chicago.)

The elimination of tidal flows for over 100 years caused by the construction of the levee in 1927 has resulted in a subsidence of the wetlands south of the levee of about 2 to 5 feet (Malamud-Roam, 1997). The most likely cause of the subsidence is the oxidation of the peat layer. In the past, the wetlands dried due to evaporation in the summer, rather than physical drainage.

Drainage from McNabney Marsh is also hindered by the railroad culvert and several pressurized liquid petroleum and natural gas pipelines, which cross under Peyton Slough just north of the railroad culvert. The railroad culvert is large but has an invert elevation of about –2 feet NGVD. This is near the elevation of much of McNabney Marsh. The elevation of the pipelines under the channel is uncertain but they are suspected to be quite shallow. Because dredging of sediment built up around the pipelines cannot be conducted due to safety issues, the Slough has not been dredged for approximately 100 feet north of the railroad culvert. The bottom elevation in the Slough around the pipelines is estimated to be approximately –1.4 to –1.9 feet NGVD.

Long periods of inundation and lack of drainage (causing stagnation due to hydraulic restrictions and subsidence) have created periods of low dissolved oxygen in McNabney Marsh during summer months, reportedly resulting in fish kills. The existing tide gate alone will not resolve the drainage issues and resulting fresh water effects to the extent anticipated because of historically caused hydraulic restrictions.

The damage from the oil spill, and the long-term lack of salt water exchange combined with long periods of fresh water inundation have caused a dramatic change in the wetland vegetation characteristics of the McNabney Marsh.

3.3.3 Analytical Methods

The operation of the tide gates and hydraulic function of the wetland areas were analyzed using the RMA2 hydrodynamic model. The RMA2 model is a two-dimensional depth-averaged finite element hydrodynamic numerical model. It computes water surface elevations and horizontal velocity components for sub-critical and free-surface flow conditions in a two-dimensional flow-field. The model has been extensively used by various agencies to simulate water levels, flow velocities, and circulation patterns in natural waterbodies such as rivers, lakes, wetlands, and estuaries and at man-made structures including bridge openings and channel reaches. A few of the model capabilities are listed below:

- Simulate both steady and transient state hydrodynamic problems, and wetting and drying conditions
- Model up to five different types of one-dimensional flow control structures such as bridge openings, culverts, and channel reaches.
- Accepts a wide variety of boundary conditions, such as velocity components by node, water surface elevations by node/line, discharge by node/line/element, tidal radiation by line, etc.

3.3.3.1 Model Inputs

The inputs to the model include geometric data, inflows at the upstream boundary, and tidal elevation data at the downstream boundary, and are described below.

Geometric Data. The area included in the analysis is shown in Figure 3.3-2 and extends along the length of Peyton Slough from the Carquinez Strait to the inflow from the Mountain View Sanitary District (MVSD). The area includes the North Marsh from just west of the existing alignment to about 300 feet east of the new alignment, Rhodia Marsh, the south spread area, Shore Terminals Marsh and McNabney Marsh. Topographic data were obtained from surveys conducted for this project for the north marsh and selected locations in other portions of the wetland. Data used to represent the topography of the other marsh areas were obtained from the CCMVCD. The following assumptions were made and selected as input for the model:

- The tidal influence in the south portion of the new alignment will be controlled by a tide gate structure consisting of five culverts with tide gates. Each culvert will have a span of 6 feet, a height of 4.5 feet, and an invert elevation of –3.5 feet NGVD. The structure was placed between the North Marsh and the south spread area.
- The dredge spoil piles will be removed down to the surrounding grade elevation and the existing Slough will be filled to the surrounding grade elevation, except between stations 24 + 00 and 35 + 00. This area will be filled to in between +3 and +4 feet NGVD.
- The south spread area will be regraded to range in elevation from approximately 0.5 to 1.75 feet.
- The elevation of the bottom of the channel will be –3.5 feet NGVD for the length of the channel, from station 0+00 to the Carquinez Strait.
- Upstream of station 0 + 00 and downstream of the railroad, the bottom of the channel will be higher because of the shallow buried pipes that run parallel to the railroad. The elevation of the pipes is unknown and the bottom of the channel at this location has not been surveyed

either as part of this study or in previous studies. Based on discussion with Karl Malamud-Roam at the CCMVCD the bottom of the channel at this location is probably between elevation 0 and –1 foot NGVD. Therefore, the channel invert was assumed to slope up over a length of 100 feet to an elevation of –1 foot to represent the higher elevation at the pipes.

- The existing culvert under the railroad was included with an invert elevation ranging from –1.4 to –1.9 feet NGVD, a span of 8 feet, and a height of 7.5 feet.

Hydraulic losses due to friction were evaluated in the RMA2 model using Manning's equation with the user-defined roughness coefficients. The Manning's "n" friction loss coefficient values used in the model were estimated to be 0.02 for the main channel and 0.12 for the wetlands. Determination of coefficients for the tide gates is described in Appendix C-1.

Input Tide. A typical tide for the Carquinez Straits at Suisun Point was synthesized from tidal data calculated from the tide predicted by WXTide (Hopper, 1999) for the year 2004. Figure 3.3-3 shows the tide used in the model.

Inflow. Inflow at the upstream boundary for non-flood conditions was assumed to be constant at 3.0 cubic feet per second (cfs), or 3.0 million gallons per day (MGD) to account for the inflow from the MVSD. This value was based on the average of the monthly average daily flows reported by the MVSD in the 2001 NPDES Compliance Summary Table. Other flows may also be present such as dry weather flows from the City of Martinez and groundwater infiltration into the channel. These values are unknown and assumed zero for this analysis.

3.3.3.2 Modeled Scenarios

Four different operational scenarios were modeled using the RMA2 model. Each scenario represents a case with a different number of tide gates operating either with two-way flow or downstream flow only. The following model inputs were common to all scenarios:

- Channel geometry as described in Section 3.3.1 above
- Same surrounding topography
- No variation of gate configuration or operation
- An average tide for Suisun Point. The tidal range achieved within the study area during the modeled period would then be roughly equivalent to the annual average daily tidal range.

Scenario 1 (0 upstream, 1 downstream)*
In this scenario, the tide gates were modeled in their existing configuration. It was assumed that all five gates would be closed to upstream flow, with only one Nekton gate allowing flow downstream. This was based on the assumption that the flap gates currently do not allow flow downstream when they are in the closed position, except during extreme flood flows.

Scenario 2 (1 upstream, 3 downstream)*
In the second scenario, the center flap gate was modeled as being fully open, allowing flow both upstream and downstream, and it was assumed that the two Nekton gates would be closed to upstream flow, but would allow flow downstream.

Scenario 3 (3 upstream, 5 downstream)*
In the third scenario, all three flap gates were modeled as being fully open, and it was assumed that the two Nekton gates would only allow flow downstream. Essentially, three gates would allow flow upstream, and five would allow flow downstream.

Scenario 4 (5 upstream, 5 downstream)*
In the fourth scenario, it was assumed that all five gates would be completely open, allowing flow both upstream and downstream.

3.3.3.3 Results of Analytical Methods

Modeling results are presented in Table 3.3-1 in terms of elevation of MLLW, MHHW, and tidal range at key locations in the marsh system.

As an example, the modeling results for Scenario 3 are shown on Figures 3.3-4a, 3.3-4b, 3.3-5a, and 3.3-5b. Figures 3.3-4a and 3.3-4b show the approximate extent of flooding north and south of the tide gates at MHHW. This represents the area that would normally get flooded at least once a day. Figures 3.3-5a and 3.3-5b show the approximate extent of flooding north and south of the tide gates at MLLW. This represents the area that generally does not drain, except possibly during spring tides.

Significant results of the modeling are:

- As more gates are opened to allow flow upstream, the tidal range increases south (upstream) of the gates, and decreases north (downstream) of the gates. The tidal range downstream of the tide gates decreases from 3.2 feet when one gate is open to 2.6 feet when five gates are open. The tidal range upstream of the gates increases from 1.4 feet when one gate is open to 1.9 feet when five gates are open.
- As more gates are opened, the mid tide level on the south side of the tide gates becomes higher.
- The greatest tidal range upstream of the tide gates will be in the south spread area with less in Rhodia Marsh, and a very small range in McNabney Marsh.

3.3.4 Conclusions and Recommendations

The cross-sectional area of the existing Slough was averaged north and south of the tide gate. Calculations indicated that the existing channel has average cross-sections of 160 ft^2 and 100 ft^2 at the marsh plain elevation north and south of the tide gate, respectively. As part of the proposed mitigation plan (see Section 4.4 Wetland Mitigation), the hydraulic capacity of the new alignment north of the tide gate will be increased by 20 percent with respect to the current alignment. This results in an increased cross-sectional area of 200 ft^2 (versus the current average of 160 ft^2), and a top width of 43 feet (versus the current average of 38 feet).

* # upstream refers to the number of fully open tide gates allowing flow upstream. # down refers to the number of gates allowing flow downstream.

The hydraulic function of the wetland areas was analyzed using the RMA2 hydrodynamic model for different scenarios of tide gate operation. Modeling results indicated that as more gates are opened to allow flow upstream, the tidal range increases south (upstream) of the gates, and decreases north (downstream) of the gates, and the mid tide level on the south side of the tide gates becomes higher. The greatest tidal range upstream of the tide gates will be in the south spread area with less in Rhodia Marsh, and a very small range in McNabney Marsh.

As shown on Table 3.3-1, the south spread area should be graded to vary between about 0 and +2 feet NGVD to obtain the greatest inter-tidal area. A wide range in elevations will allow greater flexibility for gate operations and still provide the desire wetland habitat.

The number of gates operated does not have a large effect on Rhodia Marsh water surface elevations because of the size of the wetland. For example, the elevation of MHHW only increases by 0.2 feet when the number of gates allowing flow upstream goes from one to five.

Downstream of the tide gates, the number of gates used could have an effect on the North Marsh since the elevation of MHHW drops as more gates are opened. The southern portion of the marsh plain might be flooded less frequently as more gates are opened.

3.4 EVALUATION OF HYDROGEOLOGIC CONDITIONS

3.4.1 Field Activities

To evaluate water levels and estimate hydraulic gradients in the study area, wells were installed to supplement the existing guard wells (GRD) and monitoring wells. A total of 15 shallow wells and five “deeper” wells were installed at the locations shown on Figure 3.4-1. All wells were installed using a hand auger and the cuttings for the “deeper” wells were logged for soil description (see boring logs and well installation logs in Appendix D-1 and D-2). The deeper wells have a 3.75-inch borehole over the screened interval and a 6-inch borehole for the grout seal. All the wells were constructed with 1-inch diameter schedule 40 PVC well pipe, with machine-cut 0.02-inch slots. Petroleum odor was detected in the soil during the installation of wells TI-1 and T1-2. Shallow wells were screened from 1 to 4.65 feet below ground surface (bgs) and the “deeper” wells were screened from 11 to 15 feet bgs. The shallow wells were screened to measure water levels in the shallow root zone and the silty clay interval below the root zone. The deeper wells were used to indicate water levels in the bay mud below the root zone.

Soil samples between the surface and 1.5 feet bgs were collected from eight of the 15 shallow well locations (5 east and three west of the existing Slough). Hand auguring was used to advance the hole to the sampling depth and a slide hammer driven split barrel sampler was used to collect the soil samples. Samples were retained in 3-inch diameter, 6-inch long brass sleeves. For shipment to the laboratory, the ends of the brass sleeves were covered by Teflon® tape and sealed with plastic end caps. Efforts to collect soil samples from the deeper (i.e., 9 to 13 foot) interval using hand tools failed because of the difficulty in auguring a 3-inch borehole to the desired depth and inability to retain the soft soils in the hand sampling tools.

Four deeper soil cores were collected at depths of 9 to 11 feet bgs from hydrogeologic core (HC) borings HC-1 through HC-4 located east of the Slough using an all-terrain drill rig to push thin walled Shelby tubes. Because of the numerous CCMVCD channels and the very soft nature of the soils in the marsh, the drill rig could only travel a few feet off the levee or the road just east of Zinc Hill (see Figure 3.4-1) for locations HC-1 and HC-2. The soil description was logged (see boring logs in Appendix D-2). The Shelby tubes are 3-inch diameter and 30 inches long, but were only pushed 24 inches to avoid compressing the samples. All samples were tested in the laboratory for horizontal hydraulic conductivity, porosity, and dry bulk density. Selected samples were also tested for vertical hydraulic conductivity (see Section 3.4.5).

Observations of water levels during pumping performed for development of the guard wells (GRD) indicated that only GRD-1 has a screened interval in a single hydrogeologic unit, and therefore was suitable for slug tests (H2OGEOL, 2001). Based on these results, slug tests were conducted solely on GRD-1.

The results from this investigation are summarized in the following sections.

3.4.2 Hydrogeology Conditions

Three hydrostratigraphic units have been identified at the project site (H2OGEOL 2001, RWQCB 2001):

- The Water Table Unit: This unit comprises the shallowest saturated zone beneath the site. The unit is most pronounced in the southern portion of the site and is comprised of fill, bay mud and peat. All guard wells and those installed during this investigation are located in this unit. The groundwater gradient in this Unit is discussed in Section 3.4.4.
- The Lower Intermediate/Peat Unit: The unit is irregularly distributed in the alluvium beneath the low-lying portions of the site. This unit is particularly prevalent beneath and adjacent to the former evaporation ponds. The unit comprises lenses of peat and peaty sands or mud deep within the alluvium of the site.
- The Bedrock Unit: This unit is encountered in consolidated and/or cemented material that underlies unconsolidated sediments and outcrops at the site. Some portions of the unit are confined while other portions are unconfined. The slope of the potentiometric surface of the groundwater within the unit is oriented to the southeast beneath the southern half of the project site and north towards Carquinez Strait beneath the northern half of the project site.

The Water Table Unit was the focus of this study. The top 15 feet of soil (maximum depth of borings in the newly installed wells) are part of the Water Table Unit. The upper 4 feet of the soil profile has abundant roots and plant debris. Wide areas of vegetation growing in the Peyton Slough marsh plain sediment deposits have produced (over many years) a shallow, variably permeable “root mat” layer of variable thickness overlying lower permeability bay mud which was encountered up to 4 feet bgs. Beneath the root mat layer on the marsh plain there is an extensive deposit of bay mud, which consists mostly of organic rich, fat silty clay with thin lenses (usually less than 2 feet thick) of peat and fine sandy clay (Appendix D-1 and D-2). The presence of this plant material in the root mat interval likely adds interconnecting macroporous structures that increase the hydraulic conductivity in comparison to a soil matrix without plant debris. For the purpose of this study, the Water Table Unit was sub-divided in two hydrogeologic units: the shallow root zone and the deeper bay mud.

3.4.3 Water Levels and Seasonal Variability

Groundwater elevations are within a few inches of the ground surface in both the North and South Peyton Marshes (Table 3.4-1). East of the Slough, the only areas in which the water table is not at or near the ground surface include the elevated dredge spoil piles and an elevated area of sandy soil in the vicinity of well GRD-1 (Figure 3.4-1). These high water table elevation conditions are not unexpected in wetlands bordering on a bay where the elevation of highest high daily tide equals or exceeds the ground surface elevation by as much as a 1 foot.

Historic water level data were analyzed to evaluate whether seasonal variability in water surface elevations exists at the site. Data for 38 monitoring wells were obtained from groundwater monitoring records (H2OGEOL, 2001) compiled since January 1988. Quarterly water levels were measured most years from 1988 through 1996. Since 1997, water levels have been collected semiannually.

The data collected over the past 10 years were divided into five groups for analysis. Two groups representing conditions near the Slough were divided into wells north of the levee (north Slough) and those south of the levee (south Slough). Wells not located along the Slough were categorized by their previously defined hydrostratigraphic unit (Water Table Unit, Lower Intermediate/Peat Unit or Bedrock Unit). The location of these wells is included in Appendix D-4.

Table 3.4-2 lists historic winter and summer water levels as well as average seasonal variations for each monitoring well. Overall, winter water levels have generally been higher than summer water levels. In addition, most of the highest water levels occurred in the winter of 1998, corresponding to the El Niño rain season.

The north Slough wells had an average seasonal variation of 0.33 feet. Average water levels for these nine wells ranged from 2.63 to 5.26 feet during winter, to 2.57 to 4.96 feet during summer. The seasonal fluctuations varied by year, with wet years (such as 1997 and 1998) having higher winter water levels than drier years. For some wells the higher winter levels during this extreme year extended into the next season.

The south Slough wells had an average seasonal variation of 0.87 feet. Average water levels for these six wells ranged from 0.37 to 4.89 feet during winter, to –0.85 to 4.65 feet during summer. Water levels in the three southernmost monitoring wells (MW-20, MW-8A and MW-25) dropped below the sea level for several summers between 1994 and 1998.

The Water Table Unit wells had an average seasonal variation of 1.64 feet. Average water levels for these 12 wells ranged from 1.20 to 12.74 feet during winter, to 0.29 to 10.43 feet during summer. The highest water levels occurred in MW-30, which is located in the northwest corner of the South Ore Body. MW-23A experienced unusual seasonal fluctuations with summer water levels higher than winter water levels from 1993 to 1998. MW-21 (south of MW-20) and MW-22 (south of the Storm Water Accumulation Pond) fluctuated similarly to the south Slough wells with summer water levels dropping just below the sea level during 1996 and 1997.

The Intermediate Unit wells had an average seasonal variation of 0.46 feet. Average water levels for these five wells ranged from 2.37 to 4.99 feet during winter, to 2.24 to 4.67 feet during summer. MW-64, MW-49 and MW-2B located in the South Ore Body area had at least twice the seasonal fluctuation as MW-50 and MW-36 located in the North Ore Body area.

The Bedrock Unit wells had an average seasonal variation of 1.69 feet. Average water levels for these six wells ranged from 3.82 to 12.97 feet during winter, to 2.61 to 10.32 feet during summer. The highest water levels occurred in MW-28, which is located in the northwest corner of the South Ore Body area adjacent to MW-30 from the Water Table Unit.

3.4.4 Hydraulic Gradient in the Water Table Unit

Water level elevations were measured several times between January 30th and February 11th 2002 to estimate the hydraulic gradient (Table 3.4-1). No systematic trend was observed in the differences in groundwater elevations between the shallow and deep wells except in three well cluster pairs. Data were considered indicative of a vertical gradient only if the variability in head was less than the head difference between the shallow and deep wells. That is, the average water level difference between shallow and deep wells had to be less than the standard deviation of the individual measurements. In some case the difference in water surface elevation between the shallow and deep wells in a cluster was due to the difference in ground surface elevation. Since most of the water surface elevations were near the ground surface these data were not considered indicative of a vertical gradient. With these criteria, the only well cluster pair locations that showed a consistent gradient were MW62 and TI-2 which had a downward gradient and MW4A which had an upward gradient. Well locations MW62 and TI-2 had a downward gradient of 0.055 and 0.079 ft/ft, respectively. Since both of these well locations are located near the Slough, this gradient may represent delayed drainage of water stored in the Slough bank from a previous high tide. Well location MW4A showed an upward groundwater gradient ranging from 0.028 to 0.056 ft/ft. These wells are located west of the Slough at a ground surface about 3 feet above the marsh plain. The vertical gradient results may be influenced by local conditions and are not likely to represent conditions on the marsh plain.

There is no consistent horizontal groundwater gradient based on the water elevation data collected between January 30th, and February 11th of 2002. In the north marsh, groundwater generally flows towards the existing Slough or the closest local drainage ditch on the north marsh, with a gradient of 0.006 feet/feet (ft/ft). On the south marsh, the gradient is unclear because much of the ground is flooded. A gradient of 0.006 ft/ft between the area west of the existing Slough and the new alignment was estimated. Therefore a value of 0.006 ft/ft was used as an overall gradient average between the marsh plain and the new alignment.

The horizontal groundwater gradients from the area west of the existing Slough into the cap (after remediation) were also calculated north and south of the levee, and in the tide gate area using the data collected from wells located west of the existing Slough. In the marsh plain north of the levee, the groundwater gradient was estimated to be 0.0025 ft/ft (well MW-62). In the marsh plain south of the levee, the gradient was estimated at 0.026 ft/ft (well MW-25). In the tide gate area, the groundwater gradient ranged between 0.024 (MW-4A) and 0.048 ft/ft (MW-19).

3.4.5 Soil Hydraulic Properties

Eight shallow (1 to 1.5 feet bgs) and four deeper (9 to 11 feet bgs) soil samples were collected for measurement of the soil hydraulic properties. Laboratory permeameter tests (ASTM Method D5084) were used to estimate the horizontal hydraulic conductivity on all the soil samples and the vertical hydraulic conductivity on 10 of the samples. Porosity and bulk density were measured on all the samples (ASTM Method D854). The results are summarized in Table 3.4-3. The laboratory results are included in Appendix D-3.

In the shallow interval (shallow root zone) the horizontal hydraulic conductivities ranged from 1.2×10^{-7} to 2.4×10^{-4} centimeters per second (cm/s) and vertical hydraulic conductivities ranged from 1.1×10^{-7} to 9.5×10^{-6} cm/s. In the deep interval (deep bay mud) hydraulic conductivities were much lower, with the horizontal hydraulic conductivities ranging from 3.6×10^{-8} to 1.3×10^{-7} cm/s and the vertical hydraulic conductivities ranging from 6.4×10^{-8} to 1.6×10^{-7} cm/s. As expected the lowest hydraulic conductivities from samples collected in the bay mud were measured on samples of the fat marine clays. The three samples from within the bay mud which exhibited higher hydraulic conductivities (i.e., TI-1, MW-3A, and MW-4A) were obtained from more coarse-grained samples (silt and silty sand).

The previously installed guard wells were screened from 3 to 15 feet bgs. Drawdown curves from water level measurements collected after well development had an inflection point at which drawdown rapidly increased except from GRD-1. The early, gradual decline in water level followed by rapid drawdown indicates that these guard wells initially draw water from the more conductive shallow zone, which can produce at a much greater rate when pumped than the deeper interval. When the shallow conductive layer tapped by these wells was dewatered, more rapid drawdown of the water in the well was evident because the lower bay mud interval has such low conductivity that essentially no water is yielded to the well (H2OGEOL, 2001). It was concluded that GRD-1 was the only well with a screen interval in a single hydrogeologic unit (deep bay mud) and was, therefore, suitable for slug tests (H2OGEOL, 2001). Furthermore, being located on a topographic high, well GRD-1 did not have standing water, condition that would interfere with a slug test.

A slug test in GRD-1 was conducted by measuring water levels before and after inserting a Troll® freestanding pressure-transducer/data-logger (Troll®) and after adding a water slug to the well. Forty-eight hours were allowed for the water level in the well to return to static after displacement caused by insertion of the transducer. The slug test was initiated by starting the data logger and immediately pouring water into the well to generate the initial displacement. A displacement device was not used because the Troll® cable did not allow the device to be placed in the 1-inch diameter well. The initial water level displacement was 1.17 feet and water levels were measured for approximately 132 minutes prior to the water levels reaching near pre-slug insertion levels (<0.01 feet difference). The KGS Method slug test analysis method (Hyder et al., 1994; HydroSOLV Inc., 2000b) for unsteady flow was used because it provided the best fit for the plotted data collected from the test. The slug test analysis was done using AQTESOLV® version 3.01 software (HydroSOLV Inc., 2000a and b). The hydraulic conductivity calculated with slug test data was 2.6×10^{-5} cm/s (see Table 3.4-3 and Appendix D-5). The Specific storage (Ss) estimated by this method is very unreliable and it was not reported.

The hydraulic conductivity estimated from the slug test analysis on well GRD-1 is greater than laboratory-measured values for the samples collected from the deeper hydrogeologic unit. The difference may be due to macro-porous structures present in the subsurface near the well which acted as preferential flow pathways. These structures are not represented in the very small volume of the soil core samples. However, since the slug test provides a more conservative estimate of the hydraulic conductivity than most of the laboratory data, it was used in the analysis presented in Section 3.5 Contaminant Transport and 3.6 Cap Design.

3.4.6 Preliminary Tidal Influence Study

A study of the effects of the surface water tidal fluctuation on the water levels on wells was conducted between January 25 and January 29, 2002. The tidal study was conducted by placing transducers in nine wells around the site and one in the Slough. Troll® freestanding pressure-transducer/data-loggers were placed in wells TI-1 shallow, TI-1 deep, TI-2 shallow, TI-2 deep, GRD-0, GRD-1, MW-4, MW-4A, and MW-62 (see Figure 3.4-1). The Troll® used to record water levels in the Slough was placed inside a "stilling" pipe which was a 1-inch diameter, 0.01-inch slot Schedule 40 PVC well casing attached to the bridge north of the existing tide gate. The purpose of the "stilling" pipe is to minimize the effects of wave action on the water levels recorded in the slough. The water levels in the wells were related to ground surface elevations by measuring the depth to water from the top of well casing at the start and termination of the test.

This study was conducted spanning the maximum tidal cycle in one-month period (NOAA/NOS, 2002). The total duration of the tidal study was roughly 91 hours. Because of the water level displacement created after the insertion of the transducer in the well, the first 1,000 minutes of data were disregarded and were not used in the evaluation.

Over the duration of the tidal study (excluding the first 1,000 minutes), the highest high water elevation in the Slough was 4.10 feet NGVD, the average high water was 2.47 feet NGVD, the average low water was –0.06 feet NGVD, and lowest low water was –0.93 feet NGVD. The shape of the tidal curve at lowest low tide suggests that water released from the tide gate at low tide increased the minimum water level in the Slough (Appendix D-6).

Influence during the full range of tidal cycle (i.e., response from highest high water through lowest low water) was only observed in wells TI-1 deep, TI-2 deep, MW-4, MW-4A, and MW-62 (Table 3.4-4; Appendix D-6). The maximum tidal range observed in any of the wells was 0.78 feet in well TI- 1 deep. This is consistent with the very low hydraulic conductivity of soils at the site and the relatively large storativity that is present in an unconfined aquifer such as is present at the site. The magnitude of the tidal effect on a well is proportional to the hydraulic conductivity of saturated material at the water table and inversely proportional to the storativity and distance from the water body to the well (Hsieh et al. 1987, Serfes 1991).

Wells GRD-0, GRD-1, TI-1 shallow, and TI-2 shallow indicated only partial tidal influence, showing tidal response only at the highest high water. These wells showed no tidal response when tidal levels were near mean water levels in the Slough. In the cases of TI-1 shallow and TI-2 shallow, this partial influence is interpreted to occur because the screened interval in these wells is near or above the elevation of all but the highest surface water levels. In the case of GRD-0 and GRD-1, the tidal effect is not likely transmitted laterally through the aquifer, but rather though vertical infiltration when the ground surface near these wells floods during peak tides.

Tidal efficiency and lag time were calculated for each well that responded to the full range of the tidal cycle (i.e. TI-1 deep, TI-2 deep, MW-4, MW-4A, and MW-62). The lag time is the time between when the peak occurs in the channel and when it occurs in the wells. The tidal efficiency is the proportion of tidal range in the well relative to the tidal range in the Slough. The lag times ranged from 7 minutes in TI-2 deep to 91 minutes for MW-4A. A lag time of –6 minutes was reported for MW-4; however, this was determined to be due to a clock error in the

Troll® in this well. The tidal efficiency ranged from 0.6 percent in well MW-4A to 7.8 percent in well TI-1 deep.

Overall, tidal fluctuations in the surface water in the Slough have minor effects in the groundwater elevations in the wells considered for this study and therefore a site-wide tidal influence study is not deemed necessary.

3.5 EVALUATION OF HYDROGEOLOGIC CONDITIONS

3.5.1 Field Activities

To evaluate water levels and estimate hydraulic gradients in the study area, wells were installed to supplement the existing guard wells (GRD) and monitoring wells. A total of 15 shallow wells and five “deeper” wells were installed at the locations shown on Figure 3.4-1. All wells were installed using a hand auger and the cuttings for the “deeper” wells were logged for soil description (see boring logs and well installation logs in Appendix D-1 and D-2). The deeper wells have a 3.75-inch borehole over the screened interval and a 6-inch borehole for the grout seal. All the wells were constructed with 1-inch diameter schedule 40 PVC well pipe, with machine-cut 0.02-inch slots. Petroleum odor was detected in the soil during the installation of wells TI-1 and T1-2. Shallow wells were screened from 1 to 4.65 feet below ground surface (bgs) and the “deeper” wells were screened from 11 to 15 feet bgs. The shallow wells were screened to measure water levels in the shallow root zone and the silty clay interval below the root zone. The deeper wells were used to indicate water levels in the bay mud below the root zone.

Soil samples between the surface and 1.5 feet bgs were collected from eight of the 15 shallow well locations (5 east and three west of the existing Slough). Hand augering was used to advance the hole to the sampling depth and a slide hammer driven split barrel sampler was used to collect the soil samples. Samples were retained in 3-inch diameter, 6-inch long brass sleeves. For shipment to the laboratory, the ends of the brass sleeves were covered by Teflon ® tape and sealed with plastic end caps. Efforts to collect soil samples from the deeper (i.e., 9 to 13 foot) interval using hand tools failed because of the difficulty in augering a 3-inch borehole to the desired depth and inability to retain the soft soils in the hand sampling tools.

Four deeper soil cores were collected at depths of 9 to 11 feet bgs from hydrogeologic core (HC) borings HC-1 through HC-4 located east of the Slough using an all-terrain drill rig to push thin walled Shelby tubes. Because of the numerous CCMVCD channels and the very soft nature of the soils in the marsh, the drill rig could only travel a few feet off the levee or the road just east of Zinc Hill (see Figure 3.4-1) for locations HC-1 and HC-2. The soil description was logged (see boring logs in Appendix D-2). The Shelby tubes are 3-inch diameter and 30 inches long, but were only pushed 24 inches to avoid compressing the samples. All samples were tested in the laboratory for horizontal hydraulic conductivity, porosity, and dry bulk density. Selected samples were also tested for vertical hydraulic conductivity (see Section 3.4.5).

Observations of water levels during pumping performed for development of the guard wells (GRD) indicated that only GRD-1 has a screened interval in a single hydrogeologic unit, and therefore was suitable for slug tests (H2OGEOL, 2001). Based on these results, slug tests were conducted solely on GRD-1.

The results from this investigation are summarized in the following sections.

3.5.2 Hydrogeology Conditions

Three hydrostratigraphic units have been identified at the project site (H2OGEOL 2001, RWQCB 2001):

- The Water Table Unit: This unit comprises the shallowest saturated zone beneath the site. The unit is most pronounced in the southern portion of the site and is comprised of fill, bay mud and peat. All guard wells and those installed during this investigation are located in this unit. The groundwater gradient in this Unit is discussed in Section 3.4.4.
- The Lower Intermediate/Peat Unit: The unit is irregularly distributed in the alluvium beneath the low-lying portions of the site. This unit is particularly prevalent beneath and adjacent to the former evaporation ponds. The unit comprises lenses of peat and peaty sands or mud deep within the alluvium of the site.
- The Bedrock Unit: This unit is encountered in consolidated and/or cemented material that underlies unconsolidated sediments and outcrops at the site. Some portions of the unit are confined while other portions are unconfined. The slope of the potentiometric surface of the groundwater within the unit is oriented to the southeast beneath the southern half of the project site and north towards Carquinez Strait beneath the northern half of the project site.

The Water Table Unit was the focus of this study. The top 15 feet of soil (maximum depth of borings in the newly installed wells) are part of the Water Table Unit. The upper 4 feet of the soil profile has abundant roots and plant debris. Wide areas of vegetation growing in the Peyton Slough marsh plain sediment deposits have produced (over many years) a shallow, variably permeable “root mat” layer of variable thickness overlying lower permeability bay mud which was encountered up to 4 feet bgs. Beneath the root mat layer on the marsh plain there is an extensive deposit of bay mud, which consists mostly of organic rich, fat silty clay with thin lenses (usually less than 2 feet thick) of peat and fine sandy clay (Appendix D-1 and D-2). The presence of this plant material in the root mat interval likely adds interconnecting macroporous structures that increase the hydraulic conductivity in comparison to a soil matrix without plant debris. For the purpose of this study, the Water Table Unit was sub-divided in two hydrogeologic units: the shallow root zone and the deeper bay mud.

3.5.3 Water Levels and Seasonal Variability

Groundwater elevations are within a few inches of the ground surface in both the North and South Peyton Marshes (Table 3.4-1). East of the Slough, the only areas in which the water table is not at or near the ground surface include the elevated dredge spoil piles and an elevated area of sandy soil in the vicinity of well GRD-1 (Figure 3.4-1). These high water table elevation conditions are not unexpected in wetlands bordering on a bay where the elevation of highest high daily tide equals or exceeds the ground surface elevation by as much as a 1 foot.

Historic water level data were analyzed to evaluate whether seasonal variability in water surface elevations exists at the site. Data for 38 monitoring wells were obtained from groundwater monitoring records (H2OGEOL, 2001) compiled since January 1988. Quarterly water levels were measured most years from 1988 through 1996. Since 1997, water levels have been collected semiannually.

The data collected over the past 10 years were divided into five groups for analysis. Two groups representing conditions near the Slough were divided into wells north of the levee (north Slough) and those south of the levee (south Slough). Wells not located along the Slough were categorized by their previously defined hydrostratigraphic unit (Water Table Unit, Lower Intermediate/Peat Unit or Bedrock Unit). The location of these wells is included in Appendix D-4.

Table 3.4-2 lists historic winter and summer water levels as well as average seasonal variations for each monitoring well. Overall, winter water levels have generally been higher than summer water levels. In addition, most of the highest water levels occurred in the winter of 1998, corresponding to the El Niño rain season.

The north Slough wells had an average seasonal variation of 0.33 feet. Average water levels for these nine wells ranged from 2.63 to 5.26 feet during winter, to 2.57 to 4.96 feet during summer. The seasonal fluctuations varied by year, with wet years (such as 1997 and 1998) having higher winter water levels than drier years. For some wells the higher winter levels during this extreme year extended into the next season.

The south Slough wells had an average seasonal variation of 0.87 feet. Average water levels for these six wells ranged from 0.37 to 4.89 feet during winter, to –0.85 to 4.65 feet during summer. Water levels in the three southernmost monitoring wells (MW-20, MW-8A and MW-25) dropped below the sea level for several summers between 1994 and 1998.

The Water Table Unit wells had an average seasonal variation of 1.64 feet. Average water levels for these 12 wells ranged from 1.20 to 12.74 feet during winter, to 0.29 to 10.43 feet during summer. The highest water levels occurred in MW-30, which is located in the northwest corner of the South Ore Body. MW-23A experienced unusual seasonal fluctuations with summer water levels higher than winter water levels from 1993 to 1998. MW-21 (south of MW-20) and MW-22 (south of the Storm Water Accumulation Pond) fluctuated similarly to the south Slough wells with summer water levels dropping just below the sea level during 1996 and 1997.

The Intermediate Unit wells had an average seasonal variation of 0.46 feet. Average water levels for these five wells ranged from 2.37 to 4.99 feet during winter, to 2.24 to 4.67 feet during summer. MW-64, MW-49 and MW-2B located in the South Ore Body area had at least twice the seasonal fluctuation as MW-50 and MW-36 located in the North Ore Body area.

The Bedrock Unit wells had an average seasonal variation of 1.69 feet. Average water levels for these six wells ranged from 3.82 to 12.97 feet during winter, to 2.61 to 10.32 feet during summer. The highest water levels occurred in MW-28, which is located in the northwest corner of the South Ore Body area adjacent to MW-30 from the Water Table Unit.

3.5.4 Hydraulic Gradient in the Water Table Unit

Water level elevations were measured several times between January 30 and February 11, 2002 to estimate the hydraulic gradient (Table 3.4-1). No systematic trend was observed in the differences in groundwater elevations between the shallow and deep wells except in three well cluster pairs. Data were considered indicative of a vertical gradient only if the variability in head was less than the head difference between the shallow and deep wells. That is, the average water level difference between shallow and deep wells had to be less than the standard deviation of the individual measurements. In some case the difference in water surface elevation between the shallow and deep wells in a cluster was due to the difference in ground surface elevation. Since most of the water surface elevations were near the ground surface these data were not considered indicative of a vertical gradient. With these criteria, the only well cluster pair locations that showed a consistent gradient were MW62 and TI-2 which had a downward gradient and MW4A which had an upward gradient. Well locations MW62 and TI-2 had a downward gradient of 0.055 and 0.079 ft/ft, respectively. Since both of these well locations are located near the Slough, this gradient may represent delayed drainage of water stored in the Slough bank from a previous high tide. Well location MW4A showed an upward groundwater gradient ranging from 0.028 to 0.056 ft/ft. These wells are located west of the Slough at a ground surface about 3 feet above the marsh plain. The vertical gradient results may be influenced by local conditions and are not likely to represent conditions on the marsh plain.

There is no consistent horizontal groundwater gradient based on the water elevation data collected between January 30th, and February 11th of 2002. In the north marsh, groundwater generally flows towards the existing Slough or the closest local drainage ditch on the north marsh, with a gradient of 0.006 feet/feet (ft/ft). On the south marsh, the gradient is unclear because much of the ground is flooded. A gradient of 0.006 ft/ft between the area west of the existing Slough and the new alignment was estimated. Therefore a value of 0.006 ft/ft was used as an overall gradient average between the marsh plain and the new alignment.

The horizontal groundwater gradients from the area west of the existing Slough into the cap (after remediation) were also calculated north and south of the levee, and in the tide gate area using the data collected from wells located west of the existing Slough. In the marsh plain north of the levee, the groundwater gradient was estimated to be 0.0025 ft/ft (well MW-62). In the marsh plain south of the levee, the gradient was estimated at 0.026 ft/ft (well MW-25). In the tide gate area, the groundwater gradient ranged between 0.024 (MW-4A) and 0.048 ft/ft (MW-19).

3.5.5 Soil Hydraulic Properties

Eight shallow (1 to 1.5 feet bgs) and four deeper (9 to 11 feet bgs) soil samples were collected for measurement of the soil hydraulic properties. Laboratory permeameter tests (ASTM Method D5084) were used to estimate the horizontal hydraulic conductivity on all the soil samples and the vertical hydraulic conductivity on 10 of the samples. Porosity and bulk density were measured on all the samples (ASTM Method D854). The results are summarized in Table 3.4-3. The laboratory results are included in Appendix D-3.

In the shallow interval (shallow root zone) the horizontal hydraulic conductivities ranged from 1.2×10^{-7} to 2.4×10^{-4} centimeters per second (cm/s) and vertical hydraulic conductivities ranged from 1.1×10^{-7} to 9.5×10^{-6} cm/s. In the deep interval (deep bay mud) hydraulic conductivities were much lower, with the horizontal hydraulic conductivities ranging from 3.6×10^{-8} to 1.3×10^{-7} cm/s and the vertical hydraulic conductivities ranging from 6.4×10^{-8} to 1.6×10^{-7} cm/s. As expected the lowest hydraulic conductivities from samples collected in the bay mud were measured on samples of the fat marine clays. The three samples from within the bay mud which exhibited higher hydraulic conductivities (i.e., TI-1, MW-3A, and MW-4A) were obtained from more coarse-grained samples (silt and silty sand).

The previously installed guard wells were screened from 3 to 15 feet bgs. Drawdown curves from water level measurements collected after well development had an inflection point at which drawdown rapidly increased except from GRD-1. The early, gradual decline in water level followed by rapid drawdown indicates that these guard wells initially draw water from the more conductive shallow zone, which can produce at a much greater rate when pumped than the deeper interval. When the shallow conductive layer tapped by these wells was dewatered, more rapid drawdown of the water in the well was evident because the lower bay mud interval has such low conductivity that essentially no water is yielded to the well (H2OGEOL, 2001). It was concluded that GRD-1 was the only well with a screen interval in a single hydrogeologic unit (deep bay mud) and was, therefore, suitable for slug tests (H2OGEOL, 2001). Furthermore, being located on a topographic high, well GRD-1 did not have standing water, condition that would interfere with a slug test.

A slug test in GRD-1 was conducted by measuring water levels before and after inserting a Troll® freestanding pressure-transducer/data-logger (Troll®) and after adding a water slug to the well. Forty-eight hours were allowed for the water level in the well to return to static after displacement caused by insertion of the transducer. The slug test was initiated by starting the data logger and immediately pouring water into the well to generate the initial displacement. A displacement device was not used because the Troll® cable did not allow the device to be placed in the 1-inch diameter well. The initial water level displacement was 1.17 feet and water levels were measured for approximately 132 minutes prior to the water levels reaching near pre-slug insertion levels (<0.01 feet difference). The KGS Method slug test analysis method (Hyder et al., 1994; HydroSOLV Inc., 2000b) for unsteady flow was used because it provided the best fit for the plotted data collected from the test. The slug test analysis was done using AQTESOLV® version 3.01 software (HydroSOLV Inc., 2000a and b). The hydraulic conductivity calculated with slug test data was 2.6×10^{-5} cm/s (see Table 3.4-3 and Appendix D-5). The Specific storage (Ss) estimated by this method is very unreliable and it was not reported.

The hydraulic conductivity estimated from the slug test analysis on well GRD-1 is greater than laboratory-measured values for the samples collected from the deeper hydrogeologic unit. The difference may be due to macro-porous structures present in the subsurface near the well which acted as preferential flow pathways. These structures are not represented in the very small volume of the soil core samples. However, since the slug test provides a more conservative estimate of the hydraulic conductivity than most of the laboratory data, it was used in the analysis presented in Section 3.5 Contaminant Transport and 3.6 Cap Design.

3.5.6 Preliminary Tidal Influence Study

A study of the effects of the surface water tidal fluctuation on the water levels on wells was conducted between January 25 and January 29, 2002. The tidal study was conducted by placing transducers in nine wells around the site and one in the Slough. Troll® freestanding pressure-transducer/data-loggers were placed in wells TI-1 shallow, TI-1 deep, TI-2 shallow, TI-2 deep, GRD-0, GRD-1, MW-4, MW-4A, and MW-62 (see Figure 3.4-1). The Troll® used to record water levels in the Slough was placed inside a "stilling" pipe which was a 1-inch diameter, 0.01-inch slot Schedule 40 PVC well casing attached to the bridge north of the existing tide gate. The purpose of the "stilling" pipe is to minimize the effects of wave action on the water levels recorded in the slough. The water levels in the wells were related to ground surface elevations by measuring the depth to water from the top of well casing at the start and termination of the test.

This study was conducted spanning the maximum tidal cycle in one-month period (NOAA/NOS, 2002). The total duration of the tidal study was roughly 91 hours. Because of the water level displacement created after the insertion of the transducer in the well, the first 1,000 minutes of data were disregarded and were not used in the evaluation.

Over the duration of the tidal study (excluding the first 1,000 minutes), the highest high water elevation in the Slough was 4.10 feet NGVD, the average high water was 2.47 feet NGVD, the average low water was –0.06 feet NGVD, and lowest low water was –0.93 feet NGVD. The shape of the tidal curve at lowest low tide suggests that water released from the tide gate at low tide increased the minimum water level in the Slough (Appendix D-6).

Influence during the full range of tidal cycle (i.e., response from highest high water through lowest low water) was only observed in wells TI-1 deep, TI-2 deep, MW-4, MW-4A, and MW-62 (Table 3.4-4; Appendix D-6). The maximum tidal range observed in any of the wells was 0.78 feet in well TI- 1 deep. This is consistent with the very low hydraulic conductivity of soils at the site and the relatively large storativity that is present in an unconfined aquifer such as is present at the site. The magnitude of the tidal effect on a well is proportional to the hydraulic conductivity of saturated material at the water table and inversely proportional to the storativity and distance from the water body to the well (Hsieh et al. 1987, Serfes 1991).

Wells GRD-0, GRD-1, TI-1 shallow, and TI-2 shallow indicated only partial tidal influence, showing tidal response only at the highest high water. These wells showed no tidal response when tidal levels were near mean water levels in the Slough. In the cases of TI-1 shallow and TI-2 shallow, this partial influence is interpreted to occur because the screened interval in these wells is near or above the elevation of all but the highest surface water levels. In the case of GRD-0 and GRD-1, the tidal effect is not likely transmitted laterally through the aquifer, but rather though vertical infiltration when the ground surface near these wells floods during peak tides.

Tidal efficiency and lag time were calculated for each well that responded to the full range of the tidal cycle (i.e. TI-1 deep, TI-2 deep, MW-4, MW-4A, and MW-62). The lag time is the time between when the peak occurs in the channel and when it occurs in the wells. The tidal efficiency is the proportion of tidal range in the well relative to the tidal range in the Slough. The lag times ranged from 7 minutes in TI-2 deep to 91 minutes for MW-4A. A lag time of –6 minutes was reported for MW-4; however, this was determined to be due to a clock error in the Troll® in this well. The tidal efficiency ranged from 0.6 percent in well MW-4A to 7.8 percent in well TI-1 deep.

Overall, tidal fluctuations in the surface water in the Slough have minor effects in the groundwater elevations in the wells considered for this study and therefore a site-wide tidal influence study is not deemed necessary.

3.6 CAP DESIGN

The purpose of the cap is to physically isolate impacted sediments in the existing Slough bottom from the aquatic and marsh environment, prevent resuspension and transport to other areas, and reduce the potential for vertical migration of dissolved contaminants. The cap would also function as a low permeability cutoff wall (hydraulic barrier) within the root mat zone to minimize horizontal migration of potentially contaminated shallow groundwater from the western side of the existing Slough to the new alignment.

Because various sections of the existing Slough have different depths available to accommodate cap material and have different ranges of COC concentrations, different approaches are necessary for different sections of the Slough. Based on habitat characteristics, Slough depth and COC concentration, the Slough was divided in 6 segments as follows (see Figure 3.6-1):

- Segment 1 is from Station 1+00 to 15+00
- Segment 2 is from Station 15+00 to 24+00
- Segment 3 is from Station 24+00 to 31+00
- Segment 4 is from Station 31+00 to 35+00
- Segment 5 is from Station 35+00 to 47+00
- Segment 6 is from Station 47+00 to the Slough mouth

The following sections describe the cap design approach; chemical and geotechnical sampling programs and results, chemical flux modeling and recommended alternatives for each Slough segment.

The cap design approach is presented in Section 3.6.1. The geotechnical sampling program and results are presented in Section 3.6.2. Section 3.6.3 presents the chemical investigation and findings, including the leaching and adsorption study. Section 3.6.4 presents the results and methodology for the analysis of the soft sediment cap physical properties, self-consolidation and subgrade consolidation settlement magnitudes and time rate of settlement. Section 3.6.5 presents the chemical flux modeling performed for design of the soft sediment cap and describes the model, input parameters, results and modeling limitations. The recommended conceptual cap designs are presented in Section 3.6.6 for the six project segments of the existing Slough. Cap Design Approach

A cap is an effective remediation tool for isolating contaminated sediments. Capping reduces the mobility of the underlying contaminated sediments, and keeps porewater from the underlying sediments from migrating vertically into the aquatic and marsh environment.

The key factors in the final design of the cap are physical and chemical isolation of contaminants, as well as restoration of the existing Slough to marsh habitat. Physical isolation acts to separate deeper sediments from burrowing benthic organisms. Chemical isolation is designed to control the short-term advective and long-term diffusive flux of copper and zinc in porewater from the sediments underlying the cap.

Migration of metals into the cap by advection may occur if consolidation of the sediments causes movement of porewater into the cap. The cap will be designed to retain porewater contamination that may migrate due to consolidation and to minimize long-term flux of contaminants caused by molecular diffusion and potential upward vertical groundwater seepage. Horizontal groundwater flow through the cap will also be evaluated. Engineering controls for potentially contaminated surface water over the top of the cap will be addressed in the final design.

Capping options evaluated were the placement of a soft sediment cap along the entire Slough, and the use of a low permeability barrier (such as an AquaBlok liner or other similar product) covered with a layer of sand for the Slough mouth.

3.6.1 Soft Sediment Cap

The use of a soft sediment cap for the existing Slough was evaluated through the use of an analytical computer model, which assesses the effectiveness of chemical containment of the cap, predicts the flux of contaminants through the cap and estimates required cap thickness for chemical containment. The model considers both diffusive and advective fluxes, concentrations of contaminants in the underlying sediments, the thickness of sediment layers, physical properties of the underlying sediments and cap material and other parameters. The site-specific physical and chemical data were used as input parameters to run the chemical flux model. Details are presented in Section 3.6.5.

3.6.2 Cap Liner at Slough Mouth

An AquaBlok liner was evaluated for the cap at the existing Slough mouth. AquaBlok is a composite-aggregate technology, comprised of pellets with an aggregate core, surrounded by clay-sized material and polymers. When hydrated, the particles form a relatively cohesive, low-permeability barrier, that physically, chemically, and hydraulically isolates impacted sediments.

AquaBlok is generally used with bentonite as the surrounding material, because bentonite expands when hydrated. Because salinity contents are greater than 3 to 5 parts per thousand (ppt), the swelling properties of the bentonite would be affected. Therefore, attapulgite would be used for this application instead of bentonite. Attapulgite has a double chain structure formed from bonds of silica tetrahedron sharing two of the four oxygens. Its particles are lathlike in shape and its cation exchange capacity is very low compared to montmorillonite clay minerals (sodium-type bentonite). AquaBlok manufactured with attapulgite is effective in high salinity conditions.

AquaBlok can be installed by conveyor, crane or barge with clamshell. Clean sediment would be placed on top of the AquaBlok layer to protect against erosion and allow establishment of marsh vegetation.

3.6.3 Geotechnical Investigation and Findings

The following sections present the geotechnical investigation and results.

3.6.3.1 Field Sampling and Analysis

Initial sediment sampling was conducted on December 19 and 21, 2001. Sampling locations were selected, based on the historical results, to provide representative site information. Access to sample locations was made possible using a small aluminum boat with an outboard motor. Shallow sediment geotechnical samples were collected at 7 locations along the existing Slough (Figure 3.6-1). At each location, the upper 2 to 2.5 feet of sediment was sampled. The samples were collected by pushing a 2-inch Shelby tube into the sediment along the bottom and sides of the existing slough using drill rod to extend the reach. Shelby tubes were pushed into the sidewall at approximately a 45-degree angle approximately 3 to 4 feet out from the edge of the sidewall. At each location, three Shelby tubes were collected. Samples RCM-1, RCM-2, RCM-3 and RCM-4 were collected in the vicinity of the tide gate. Sample RCM-1 was collected at the bottom of the channel. Sample RCM-2 was collected also from the bottom of the channel, with RCM-3 and RCM-4 collected from the west and east sidewalls near RCM-2, respectively. Also at RCM-4, two 5-gallon buckets of sediment were collected from the channel bottom.

Farther north along the Slough alignment, samples RCM-5, RCM-6, and RCM-7 were gathered as shown on Figure 3.6-1. The RCM-5 tube was collected from the bottom of the channel, the RCM-6 tube was collected from the east sidewall of the channel, and the RCM-7 tube was collected from the west sidewall.

Sediment recovered from the tubes pushed into the bottom and sidewalls of the existing channel was retained in the original Shelby tubes. The tubes were cut to the length of the sample (about 2 feet) and then cut into two samples separating the top 1 foot and the bottom 1 foot of the sample. The ends of the cut sample barrels were sealed at both ends with plastic caps and vinyl tape and sent to the URS geotechnical laboratory in Pleasant Hill, CA for testing.

Two hand augers holes were drilled along the new alignment at two locations to a depth of 5 feet bgs. NSCM-1 was located at Station 35+00 and NSCM-2 was located at Station 18+80 (Figure 3.6-1). These were sampled by pushing Shelby tubes with a drill rod and letting them sit for a few minutes so that the soft sediment would not slip out of the tube. The top 2 feet were discarded and the rest of the sediment from 2 to 5 feet was mixed together in a 5-gallon bucket and sent to URS geotechnical laboratory in Pleasant Hill, CA for testing.

The sample locations were chosen to provide physical data for general cap design and for the chemical flux modeling. The tube samples were analyzed for density, moisture content, porosity, specific gravity, Atterberg limits, hydrometer and grain size distribution. The two composite samples were analyzed for moisture content, specific gravity, Atterberg limits, hydrometer, and grain size distribution.

Four deep geotechnical boreholes were drilled and sampled during the period from December 12 to December 27, 2001. The description of the drilling equipment, drilling methods, and sampling tools used are presented in Section 3.7 and Appendix F. The locations of the four boreholes are presented in Figure 2 in Appendix F. Boreholes URS-1, URS-2, URS-3, and URS-4 were advanced to depths of 100.8, 81.0, 52.5, and 61.5 feet, respectively.

3.6.3.2 Results

The results from the four geotechnical boreholes are presented on the log of borings presented in Appendix F) and are shown on a generalized geologic profile in Figure 3 in Appendix F.

Physical Parameter Results

Test results for the shallow sediment samples are presented in Table 3.6-1. The samples were predominantly dark gray clays and silts (bay mud) with varying amounts of organics, including layers of peat. The bottom samples (RCM-1, 2, and 5) tended to be finer grained, with percentages of sand sized particles ranging from 4 to 63 percent. The sample with 63 percent sand sized particles (RCM-2-2), however, was predominantly peat. Porosity ranged from 72.2 to 89.7 percent and specific gravity from 2.13 to 2.77.

The samples from the sidewall (RCM-3, 4, 6, and 7) had percent sand sized particles ranging from 7 to 64 percent. RCM-3 and 4 were predominantly peat at both the 0 to 1 and 1 to 2 foot depths. Porosity ranged from 75.8 to 89.6 percent and specific gravity from 2.15 to 2.71.

The laboratory tests performed on the samples recovered from the geotechnical boreholes included: moisture content, unit weight, specific gravity, Atterberg limits, gradation, unconfined compression, unconsolidated-undrained triaxial compression, and consolidation.

The tests were performed by the URS Soils Laboratory in Pleasant Hill and the results are summarized in Table B-1 in Appendix F. The testing program and procedures used for the different types of tests are briefly described in Appendix F. Also presented in Appendix F are the individual laboratory data reports.

Consolidation Test Results

Fourteen consolidation tests were performed on samples of organic clay, fat clay, lean clay, and sandy fat clay that were obtained from borings URS-1, URS-2, URS-, and URS-4, and from sample RCM-1. The consolidation tests were conducted to evaluate the soil's compressibility characteristics and influence of past geologic history. All consolidation tests were performed in general accordance with the procedures outlined in the ASTM D2435 Standard.

Table 3.6-2 presents the results of the consolidation tests. For each test, the table summarizes the moisture content, unit weight, in-situ effective stress, maximum past pressure, overconsolidation ratio (OCR), the compression ratio (CR), recompression ratio (RR), coefficient of consolidation during virgin compression ($c_{v(virgin)}$) and coefficient of consolidation during recompression ($c_{v(rec)}$).

The in-situ effective stress was calculated by assuming a unit weight based on laboratory tests. The maximum past pressure was estimated using the Casagrande construction and the end-of-primary consolidation curve. The OCR was computed by dividing the maximum past pressure by the in-situ effective stress. The compressibility parameters, CR and RR, were obtained from the end-of-primary consolidation curve. The CR and recompression ratio represent the vertical strain per log cycle of stress during virgin compression and recompression, respectively. Using the square root of time method, the vertical c_v was calculated for $c_{v(virgin)}$ and for recompression $c_{v(rec)}$.

An evaluation of the test results indicates that the soil in each of the four borings is overconsolidated. Generally, the OCR is high near the ground surface and decreases with increasing depth. From 0 to 5 feet depth, the OCR is greater than 7. In borings URS-1 and URS-2, the OCR decreases to about 2 at approximately 25 feet. The results from boring URS-3 do not show a definitive trend, and we suspect that the presence of abundant shells in some of the URS-3 samples may have inhibited an appropriate characterization of the compressibility. In boring URS-4, the OCR decreases to about 2 at a depth of 40 feet.

The CR ranged from 0.24 to 0.31 for the uppermost clay layer, generally the upper 4 to 9 feet, in all of the borings and sample RCM-1 with the exception of boring URS-3. In URS-3, samples at 5 to 7.5 feet and 10 to 12.5 feet depth revealed relatively low CRs (0.19 and 0.13, respectively), which can be attributed to the presence of sand and shells in these samples. The peaty clay layer, which lies below the uppermost clay layer, exhibited much higher CRs, ranging from 0.33 to

0.56. For two samples tested in the deeper clay layer at about 40 feet, the CR ranged from 0.20 to 0.22.

The RRs show trends that are similar to those of the compression ratios (CR). The RR ranges from 0.017 to 0.046 in the uppermost clay layer, and the RR is much higher, ranging from 0.031 to 0.087 in the peaty clay layer. In the layer of sandy clay with shells from URS-3, the RR ranges from 0.015 to 0.018, which is not substantially different from the RR in the uppermost clay layer of the other borings and sample RCM-1. For the lower clay layers at 40 feet, the RR ranged from 0.027 to 0.047.

The uppermost clay layer exhibited low coefficients of consolidation (c_v). In the virgin compression zone, the $c_{v(virgin)}$ ranges from 0.0356 to 0.0490 ft^2/day. For the sandy clay layer in URS-3, the $c_{v(virgin)}$ was approximately 2.16 ft^2/day. For the peaty layers, the $c_{v(virgin)}$ was slightly higher, ranging from 0.0356 to 0.0862 ft^2/day.

For the upper clay recompression zone, the $c_{v(rec)}$ ranges from 0.0119 to 0.0779 ft^2/day and for the sandy clay layer, the $c_{v(rec)}$ was 1.915 ft^2/day. For the peaty layer, the $c_{v(rec)}$ ranged from 1.850 to 0.4530 ft^2/day. For the lower clay specimens at 40 feet, the $c_{v(rec)}$ was 0.6755 ft^2/day.

These results were used in Section 3.6.4. In general, for each individual consolidation test specimen, the c_v decreased as the specimen was consolidated from the recompression zone ($c_{v(rec)}$) to the virgin zone ($c_{v(virgin)}$). The higher coefficients of consolidation for the sandy clay can be attributed to the higher permeability in this material. In some tests, the $c_{v(rec)}$ prior to the maximum past pressure was lower than the $c_{v(virgin)}$ just following the maximum past pressure. This may be due to the difficulty associated with obtaining accurate c_v data during recompression because of sample disturbance or shell and/or organic matter affecting the permeability during a specific stress interval. However, even for those tests that showed these particular anomalies, the ultimate trend was a decreasing $c_{v(virgin)}$ following the maximum past pressure.

3.6.4 Chemical Investigation and Findings

Site-specific physical and chemical data were collected to design a sediment cap to isolate contaminated sediments in the existing Slough and provide the design parameters and criteria. Samples were collected from the Slough sediments, potential cap material, and groundwater. Sediment porewater samples were also generated from selected sediment samples. Chemical and physical analyses, and leaching and adsorption tests were conducted with these samples. The field activities, analyses, and data quality are described below. The chemical laboratory reports are included in Volume II of this report.

3.6.4.1 Sediment and Porewater

Sediment Sampling

Initial sediment sampling was conducted on December 19 and 21, 2001. The samples were collected in conjunction with the geotechnical sampling at the same seven locations (RCM-1 through RCM-7). There were three sample locations in the Slough bottom and four in the sidewall. For each boring, 0 to 1 foot and 1 to 2 feet depth intervals were sampled. The sample locations are shown on Figure 3.6-1. The tubes were then capped at both ends, and cut and capped into 1-foot lengths taking care to avoid headspace inside the tube. The samples were then labeled, taped around the caps, and stored in an ice chest with ice. Proper chain-of-custody protocol was followed during sample collection. Additional sampling was conducted on February 5, 2002. The purpose was to sample an area with higher copper and zinc concentrations. Three locations were sampled (RCM-8 through RCM-10).

In addition, two composite sediment samples (NSCM-1 and NSCM-2) were taken from the 2 to 5 feet of the new alignment area for evaluation of suitability as potential cap material and for leaching and adsorption studies.

Porewater Generation

Porewater samples were generated from the RCM sediment samples by centrifuging the sediment samples at 2,500 rpm under anaerobic conditions. Sufficient sediment was centrifuged to obtain at least 500 mL for each porewater sample. Fifty percent of the porewater volume per sample was filtered through a 0.45 μm size filter and collected for dissolved metals and organic carbon analyses.

Sediment Analysis and Results

Sediment samples from the initial sampling (14 RCM samples and 2 NSCM samples) were analyzed for physical properties of grain size, density, specific gravity, moisture content, Atterberg limits, and hydrometer by the URS geotechnical laboratory in Pleasant Hill, CA. The chemical analyses include Total Organic Carbon (TOC) (EPA Method 9060), metals (copper and zinc) (EPA Method 6010), moisture content (EPA Method 2540G), pH (EPA Method 9045), and acid volatile sulfide (AVS) – simultaneously extracted metals (SEM) (copper and zinc only) (EPA Method 200.0/245). Three samples (RCM-1-1, RCM-2-1, and RCM-5-1) were analyzed for chloride. Four samples (RCM-1-1, RCM-2-1, RCM-9, and RCM-10) were analyzed for Toxicity Characteristic Leaching Procedure (TCLP) for copper and zinc. Two samples from the New Alignment area (NSCM-1 and NSCM-2) were analyzed for 10 metals, pesticides, polychlorinated biphenyls (PCBs), semi-volatile organic compounds (SVOCs), and general chemistry. Samples for chemical analyses were sent to Columbia Analytical Services, Inc. of Kelso, Washington on a standard turnaround time.

The second round of sediment sampling was conducted in an area of historically elevated metals concentration. The lab received the three samples and did an initial metal screening. The two samples with the highest results (RCM-9 and RCM-10) were then analyzed for general chemistry and copper and zinc by the methods above.

Existing Slough

Results of the sediment testing are presented in Table 3.6-3. Copper concentrations in the sediment samples, measured by EPA Method 6010, ranged from 33 to 93,200 mg/kg, with an average of 11,004 mg/kg. Concentrations tended to be higher in the samples from the Slough bottom, ranging from 220 to 93,200 mg/kg, and averaging 21,559 mg/kg. Sidewall sample copper concentrations ranged from 33 to 2,520 mg/kg and averaged 448 mg/kg.

Zinc concentrations in the sediment samples, measured by Method 6010, ranged from 327 to 36,300 mg/kg for all samples. The average concentration for all samples was 6,047 mg/kg.

Bottom sample concentrations ranged from 347 to 36,300 mg/kg and averaged 10,833 mg/kg. Sidewall sample concentrations ranged from 327 to 3,780 mg/kg and averaged 1,262 mg/kg.

AVS concentrations ranged from 81 to 5,920 mg/kg for the samples analyzed. Bottom samples had concentrations from 122 to 5,920 mg/kg and averaged 2,282 mg/kg. Sidewall samples ranged from 81 to 5,570 mg/kg and averaged 1,107 mg/kg.

Copper in SEM ranged from 2 to 20,700 mg/kg, and averaged 3,167 mg/kg. Bottom sample concentrations for copper ranged from 119 to 20,700 mg/kg, averaging 5,839 mg/kg. Sidewall sample concentrations ranged from 2 to 3,220 mg/kg, and averaged 496 mg/kg.

Zinc in SEM ranged from 224 to 32,800 mg/kg for all samples and averaged 5,399 mg/kg. For bottom samples, zinc concentrations ranged from 272 to 32,800 mg/kg and averaged 9,489 mg/kg. Sidewall sampled ranged from 224 to 1,880 mg/kg, averaging 1,308 mg/kg.

TOC ranged from 1.5 to 16.9 percent with average of 6.44 percent. Solids ranged from 19.8 percent to 51.4 percent with average of 36.9 percent. Neither parameter showed significant differences between sidewall and bottom samples.

Sediment pH ranged from 6.1 to 7.9 for all samples, averaging 7.5. In bottom samples, pH ranged from 6.1 to 7.8 and averaged 7.3. Sidewall sample pH ranged from 7.2 to 7.9 and averaged 7.6.

The metal concentrations in the sediments were in general much higher in the Slough bottom than the sidewall. In average concentrations, copper was about 48 times higher and zinc was about 8.6 times higher in the Slough bottom sediment than in the sidewall sediment.

New Alignment

Results of the New Alignment sediment testing for two samples NSCM-1 and NSCM-2 are presented in Table 3.6-4. Sediment pHs were 7.8 and 6.8. TOC values were 1.6 percent and 12.2 percent. Solid contents were 43.9 percent and 23.5 percent.

Copper concentrations for two samples varied at 120 and 59 mg/kg, while zinc concentrations were relatively similar at 224 and 206 mg/kg.

AVS concentrations were 884 and 63.9 mg/kg. In SEM, copper concentrations were 127 and 7.4 mg/kg, and zinc concentrations were 148 and 146 mg/kg.

Pesticide results were mostly non-detect, with the exception of beta-BHC concentration of 0.45 μg/kg for NSCM-1 and methoxychlor concentration of 7.9 μg/kg for NSCM-2. PCB results were all non-detect for both samples tested. SVOCs were detected at low concentrations. All SVOC concentrations were below the screening value for beneficial reuse of dredged material for wetland surface material (RWQCB, 2000).

All metal concentrations for new alignment samples were below the ER-M levels. Metal concentrations were also compared to the background metal concentrations derived in Section 3.5.2 for the project site (60 mg/kg for copper and 169 mg/kg for zinc). Sample NSCM-1 had concentrations higher than the background for both copper and zinc. Sample NSCM-2 had zinc concentration higher than background, but copper concentration below background. A leaching test was conducted with sample NSCM-1 to evaluate the leachability of the metals. The results are used to assess the usability of the sediment as potential cap material. This study is presented in Section 3.6.3.3.

Summary of Existing Data

Figures 3.6-2 and 3.6-3 and Table 3.6-5 present the data used for cap design. Sediment samples from the Slough bottom concentrations ranged from 71 to 452,000 mg/kg for copper and 158 to 93,200 mg/kg for zinc. Concentrations in samples from the west sidewall had ranged from 20 to 2,424 mg/kg copper and 79 to 13,853 mg/kg zinc. East sidewall sample concentrations ranged from 36 to 4,329 mg/kg copper and 96 to 4,175 mg/kg zinc

In Segment 1, copper in Slough bottom sediments ranged from 156 to 3,200 mg/kg, while zinc ranged from 398 to 3,810 mg/kg. Along the west sidewall concentrations ranged from 316 to 476 mg/kg for copper and 186 to 2,857 mg/kg for zinc. East sidewall samples ranged from 693 to 866 mg/kg for copper and 519 to 1,558 mg/kg for zinc.

For Segment 2, bottom sediments had 220 to 53,900 mg/kg copper and 347 to 25,100 mg/kg zinc. West sidewall samples ranged from 32.5 to 2,424 mg/kg copper and from 433 to 2,208 mg/kg zinc. East sidewall samples ranged from 61.3 to 4,329 mg/kg copper and 327 to 2,251 mg/kg zinc.

In Segment 3, copper in Slough bottom sediments ranged from 865 to 121,000 mg/kg, while zinc ranged from 6,940 to 88,300 mg/kg. Along the west sidewall concentrations ranged from 39 to 2,500 mg/kg for copper and 270 to 13,853 mg/kg for zinc. East sidewall samples ranged from 43 to 950 mg/kg for copper and 110 to 3,180 mg/kg for zinc.

In Segment 4, copper in Slough bottom sediments ranged from 2,980 to 452,000 mg/kg, while zinc ranged from 1,220 to 93,200 mg/kg. Along the west sidewall a single sample had 157 and 501mg/kg for copper and zinc respectively. East sidewall samples ranged from 36 to 2,714 mg/kg for copper and 96 to 4,175 mg/kg for zinc.

In Segment 5, copper in Slough bottom sediments ranged from 85 to 10,300 mg/kg, while zinc ranged from 158 to 7,260 mg/kg. West sidewall sample concentrations ranged from 20 to 4,000 mg/kg for copper and 79 to 4,000 mg/kg for zinc. East sidewall sample concentrations ranged from 138 to 814 mg/kg for copper and 205 to 2,923 mg/kg for zinc.

In Segment 6, copper in Slough bottom sediments ranged from 71 to 1,610 mg/kg, while zinc ranged from 192 to 2,120 mg/kg. A single sample from the west sidewall had 171 and 689 mg/kg for copper and zinc respectively. No samples have been taken in the east sidewall.

Porewater Analysis and Results

Sediment porewater samples were generated from the RCM sediment samples by centrifuging the sediment samples. Porewater samples were analyzed for general chemistry (TOC, pH, salinity) and copper and zinc. A portion of the porewater from each sample was also filtered through 0.45 μm filter for analyses of dissolved copper, zinc, and TOC.

Porewater chemistry results are presented in Table 3.6-6. Porewater pH ranged from 6.8 to 8.4 for all samples, averaging 7.7. In bottom samples, pH ranged from 6.8 to 8.0 and averaged 7.6. Sidewall sample pH ranged from 7.5 to 8.4 and averaged 7.9.

Total copper in porewater ranged from <3.0 to 1,450 μg/L in all samples. Copper concentrations averaged 230 μg/L. In bottom samples, copper ranged from 5 to 1,450 μg/L and averaged 440 μg/L. Sidewall sample concentrations ranged from <3.0 to 58 μg/L and averaged 20 μg/L.

Total zinc in porewater ranged from 10 to 166,000 μg/L for all samples and averaged 15,232 μg/L. For bottom samples, total zinc concentrations ranged from 29 to 166,000 μg/L and averaged 30,342 μg/L. Sidewall sample concentrations ranged from 10 to 301 μg/L and averaged 121 μg/L.

In comparison, the Slough bottom had much higher porewater metal concentrations than the Slough sidewall. The average concentration for the Slough bottom was 22 times higher for copper, and 251 times higher for zinc. The sidewall was less contaminated than the bottom. Thus, there would be less chemical migration from the sidewall area into the environment.

Correlation of Sediment/Porewater Concentrations

Potential correlations between porewater concentrations and associated sediment sample concentrations were evaluated. However, there was no simple correlation for metal (copper and zinc) concentrations. This may be due to relatively high and variable sulfide contents in the sediment samples. Sulfide reacts with copper and zinc forming metal sulfide compounds with very low solubilities, removing the metals from solution.

3.6.4.2 Groundwater

Site groundwater flows from west to east. Therefore, water from a monitoring well from west of the Slough (MW-8A) was selected for the sequential batch leaching test and adsorption test. The field groundwater sampling and analysis and results are described below.

Field Sampling

The site groundwater sample was collected from the monitoring well MW-8A (Figure 3.4-1) on January 23, 2002. The well was purged using standard methods whereby a minimum of three well casing volumes (or until the well is purged dry) is removed from the well while monitoring pH, conductivity and temperature. Prior to purging, the groundwater level in the well was measured and recorded. In the case of MW-8, the well was purged dry after approximately 12 gallons of the groundwater were removed.

Samples were then collected from the well into laboratory supplied 40-mL volatile organic analysis (VOA) vials containing hydrogen sulfide, 250-mL plastic bottles preserved with nitric acid and two specially designed 10-liter bags containing nitrogen gas to maintain the sample in an anaerobic environment. The VOA vials and plastic bottles were filled to zero headspace while the 10-liter bags were filled to near capacity. Insufficient sample water was obtained to fill both 10-liter bags on the same day due to the slow groundwater recharge.

The well was purged again on January 30, 2002 and additional sample was obtained to fill the second 10-liter bag. After collection, the samples were labeled and stored in an ice chest with ice. Proper chain-of-custody protocol was followed during sample collection and shipment.

Groundwater Analysis and Results

Groundwater samples were analyzed for TOC and copper and zinc (total) using EPA Method 415.2 and 6010b, respectively, by Curtis and Tompkins, Ltd. of Berkeley, California on a 10-day turnaround time. The specially designed 10-liter bags were sent to S&S Environmental Applications of Bainbridge Island, Washington for Adsorption and Leaching Tests based on the ACE methods.

No copper and zinc were detected in the groundwater sample. TOC was 32 mg/L, pH was 7.07, redox potential was –225 mV, and conductivity was 18.9 mMho.

3.6.4.3 Leaching and Adsorption Studies

A sequential batch leaching test was conducted to evaluate the leaching potential of the cap material with low-level concentrations of metals. Adsorption testing was conducted to evaluate the adsorptive capacity of the cap material and to obtain the metal partition coefficients required for the chemical flux model. Both tests were run at Soil Technology Inc. in Bainbridge Island, Washington. The leaching and adsorption tests were conducted using site groundwater collected from monitoring well MW-8A, and two sediment samples collected from the new alignment area (NSCM-1 and NSCM-2).

The sediment sample (NSCM-1) with higher metal concentrations (Cu = 120 mg/kg, Zn = 224 mg/kg) was used for the leaching study, to evaluate how much, if any, of the metals in the sediment could leach into the groundwater (Table 3.6-4).

The sample with the lower concentration of metals (NSCM-2) was used in the adsorption test to evaluate the level of metals that could be bound within the cap material. From the adsorption ratio, metal partition coefficients in the cap were calculated for use in the flux model.

High sulfide content was noted in the groundwater sample (hydrogen sulfide smell), but metals were not detected. In fact, when the lab added copper to the original solution precipitation of CuS occurred. However, when the lab adjusted the groundwater pH from the original 7.07 down to 4.5, copper was then detected at concentrations ranging from 400 μg/L to 770 μg/L, but zinc was still not detected. This suggests that pH and sulfide are very important for metal solubility. The relationship between pH, sulfide and metal solubility for the entire project site is further evaluated in Section 3.5.3.

Neither copper nor zinc were detected in the leachates from the sequential batch leaching test which used the groundwater as leaching solution. The lab also ran a control test using deionized water as the leaching solution and neither copper nor zinc were detected in these leachates. However, after acidification of the leachate to pH 4.5, copper was measured at 780 μg/L, but no measurable level of zinc was detected in the DI water leachate. The leaching results suggest that material from the new alignment could accommodate low-level metal contamination when pH is near neutral and high sulfide concentrations are present.

During the adsorption test, equilibrium time was first determined. The equilibration time was low (less than 25 minutes). The low equilibrium time was expected based on the anticipated precipitation reaction with the sulfide. The adsorption coefficient was calculated using the Freundlich equation:

$$\frac{x}{m} = C_s = KC_e^{1/n}$$

where:

m = grams of mass of sediment

x = micrograms of Zn (or Cu) adsorbed by m grams of sediment

C_s =equilibrium concentration of Zn (or Cu) in the sediment

C_e = equilibrium concentration of Zn (or Cu) in the solution

K= Freundlich adsorption constant

l/n = exponent where n is a constant

The data are provided in Exhibit 3.6-1 below.

Exhibit 3.6-1
Adsorption Test Results

	Freundlich Adsorption Constant (K)	Constant (n)
Cu	22,000	5.0
Zn	4,800	2.5

These results were used to calculate the site-specific partition coefficients of copper and zinc in the cap material for input parameter for the chemical flux model.

3.6.4.4 Data Validation and QA/QC

Analytical laboratory results were evaluated to assess the quality of individual sample results and overall method performance. Analytical performance was evaluated on both an individual sample and a quality control batch (groups of samples prepared and analyzed together) basis. The data evaluation performed included review of:

- Blanks (method and trip blanks);
- Spikes (surrogate, laboratory control, and matrix spikes);
- Sample Integrity (chain-of-custody documentation, sample preservation, and holding time compliance).

The accuracy and precision of the data were found to be acceptable for use of these data in project decisions with the qualifications summarized in Table 3.6-7. The detailed QA/QC is included in Appendix B-3.

3.6.5 Geotechnical Analysis of the Cap

The geotechnical analysis performed for the cap design consisted of the following

- Estimating the installed physical properties of the soft sediment cap
- Predicting the self consolidation of the soft sediment cap
- Predicting the primary consolidation of subgrade soils due to cap/liner fill weight
- Predicting the time for self consolidation and subgrade consolidation to occur

3.6.5.1 Soft Sediment Cap Properties and Consolidation

For this report, material from the new alignment and import from an off-site borrow source were considered. The physical and compressibility properties for the new alignment source are estimated from the results of laboratory index tests performed on samples recovered from borings NCSM-1 and NCSM-2 and approximate correlations for consolidation characteristics of silts and clays (U.S. Department of the Navy, 1982). The physical and compressibility properties of the import borrow [fat clay (CH)] are estimated from typical values presented in U.S. Department of the Navy (1982). These properties are summarized in Table 3.6-8.

Prediction of self-consolidation for three different cap materials was computed for varying cap thicknesses. Computations were performed for 3-, 4-, and 6-foot-thick caps. The low-density organic silts/clays (MH) exhibited self-consolidation settlement values that are more than two times those of the fat clays (CH). Therefore, the low-density organic silts/clays (MH) were eliminated from consolidation for the cap design. The amount of predicted self-consolidation 90 and 105 pcf total density (unit weight) for the different cap material and thickness are presented in Table 3.6-9.

The time required for about 95 percent of self-consolidation to occur could range from 360 to 12,000 days, depending on the cap material type and thickness. The time-rate estimates are based on singled drainage conditions. The estimated degree of consolidation 30 days after placement could range from less than 2.5 to about 20 percent, depending on the cap material type and thickness. The estimated time for 95 percent consolidation and degree of consolidation after 30 days are presented in Table 3.6-9.

3.6.5.2 Subgrade Consolidation Settlement Analysis

Introduction

This section presents the methodology and results of the consolidation settlement analysis. Table 3.6-10 presents the estimated consolidation settlement of the subgrade soils beneath the cap for two different fill total unit weights. Table 3.6-11 presents the estimated total amount of settlement (i.e., self consolidation of cap and consolidation of the subsoils) that will occur 1 year after construction of the cap. Table 3.6-11 also lists the estimated time required for 95 percent consolidation.

Placement of the cap would apply a vertical load to the subgrade soils, which consist of peaty organic clays, lean clays, and fat clays. The application of the vertical load initially would generate excess pore pressures in the clay strata. With time, the excess pore pressure would dissipate as the added stress is transferred from the pore fluid to the mineral skeleton. During this transfer of stress, the clay strata would undergo volume changes. The rate of volume change and corresponding settlement is governed by how fast the pore fluid can drain out of the clay under the induced hydraulic gradients. This gradual process is called primary consolidation. The following sections describe the methodology used to estimate consolidation settlement and time rate of consolidation settlement.

The consolidation settlement analysis methodology can be divided into three major sections: station segment and cap geometry, consolidation settlement analysis, and time rate of consolidation analysis.

Station Zone and Cap Geometry

Table 3.6-10 lists a summary of the estimated consolidation settlements for six segments and the tide gate area along the cap alignment. The number of zones and length of each zone was based on the ground surface profile and the proximity to a particular boring. For the stations within each zone, the ground surface profile is similar and the chosen boring represents the nearest one available. The following Exhibit 3.6-2 lists the six segments and tide gate area, and the associated boring profile:

Exhibit 3.6-2

Segment Summary

Segment	Station	Boring
1	1+00 to 15+00	URS1
2A	15+00 to 20+00	URS1
2B	20+00 to 24+00	URS2,RCM1
3A	24+00 to 26+00	URS2,RCM1
3B	26+00 to 30+00	URS2,RCM1
Tide Gate Area	About 31+00	URS
4	32+00 to 35+00	URS4
5A	35+00 to 37+00	URS4
5B	37+00 to 47+00	URS4
6	47+00 to 53+00	URS3

In addition to defining the subgrade soil profile for each station zone, cap geometry and composition were defined. The exact cap geometry depends upon the cap thickness and the ground surface profile. For each zone, one cap geometry with a triangular cross section was assumed, with the apex at elevation of –3 feet NGVD and the long edge varying from elevation +1 to +4 feet, depending on the cap thickness and final grade.

For cap composition, two different fill material total unit weights were assumed: 90 and 105 pounds per cubic foot (pcf). It was also assumed that the groundwater level was at the top of the soft sediment cap in most cases. Exceptions were segments 3B, tide gate area, and segment 4 where the groundwater level was estimated to be 1 to 2 feet below the final grade.

As listed in Table 3.6-10, the estimated settlement is given at three different locations along the profile of the cap: the west edge, centerline, and the east edge.

As an example of the variables used in station zoning and cap geometry, consider Station 1+00 to 15+00 in Table 3.6-10. A cap thickness of 3 feet at the centerline of the Slough was assumed. Based on the individual station profiles from Station 1+00 to 15+00, a cap geometry with a triangular cross section with a long edge width of 50 feet at elevation +1.0 feet and the apex at elevation –3 feet was assumed.

Consolidation Settlement Analysis

To estimate consolidation settlements, the following general equations were used:

$$\rho_{cf} = \Sigma h[RR*\log(\sigma'_p/\sigma'_{vi}) + CR*\log(\sigma'_{vf}/\sigma'_{vp})] \qquad (1)$$

$$\rho_{cf} = \Sigma\, h[RR*\log(\sigma'_{vf}/\sigma'_{vi})] \qquad (2)$$

where: ρ_{cf} = final consolidation settlement

h = height of clay layer

RR = recompression ratio

CR = compression ratio

σ'_p = maximum past pressure

σ'_{vi} = initial effective stress

σ'_{vf} = final effective stress

In order to use this equation to estimate consolidation settlements for the entire clay stratum for each station zone, a three-step methodology was used:

1. Evaluation of the consolidation parameters.
2. Development of stress history, initial effective stress, and final effective stress profiles.
3. Computation of settlement using consolidation settlement equation.

The consolidation parameters, including RR, CR, and σ'_p in the equation above, were evaluated based on fourteen consolidation tests performed on soil samples obtained from borings URS-1, URS-2, URS-3, URS-4 and sample RCM-1. Table 3.6-2 lists the results of the consolidation tests, and Section 3.6.3.2 describes the procedures used to obtain the consolidation parameters.

For each station zone, a stress history, initial effective stress, and final effective stress versus depth profiles were developed. The stress history profile (σ'_p vs. depth) was obtained by plotting the maximum past pressure versus depth. The initial effective stress profile (σ'_{vi} vs. depth) was calculated assuming the water table at the top of the soft sediment cap, and assigning a unit weight to each clay stratum based on laboratory tests. The initial effective stress profile is the in-situ effective stress prior to placement of the cap.

The final effective stress profile (σ'_{vf} Vs depth) represents the soil stratum effective stress following cap placement, assuming all excess pore pressure has dissipated. The final effective stress profile represents 100 percent consolidation. To estimate the distribution of applied stresses in the clay stratum, it was assumed that the cap placed a symmetrical vertical triangular loading upon the ground surface. For this loading, elastic solutions were used (Poulos and Davis, 1974) to determine the applied stresses beneath the centerline, west edge, and east edge of the cap. The applied stress was added to the initial effective stress to estimate the final effective stress.

To estimate consolidation settlements, it was assumed that the clay strata would undergo one-dimensional consolidation. That is, all of the volume change due to consolidation would be due to vertical strain with no lateral deformation. This assumption allows use of the parameters obtained from the odometer consolidation test results (Table 3.6-2). In reality, consolidation volume change includes both vertical and lateral deformation. Given the Slough cap loading condition and subgrade soils stress history, the one dimensional consolidation assumption will lead to slightly overestimated consolidation settlements.

The entire series of clay strata was divided into 1-foot intervals and settlement was computed for each layer using the equations presented above. For simplicity, it was assumed that the soil in each layer had an OCR equal to 2. Since the upper 20 to 25 feet of clay has an OCR greater than 2, this assumption leads to a slight overestimation of the consolidation settlements.

Equation 1 was used if the final vertical stress (σ'_{vf}) exceeded the maximum past pressure (σ'_p), and Equation 2 was used if the final vertical stress (σ'_{vf}) was less than the maximum past pressure (σ'_p).

The settlement from each 1-foot layer was summed over the whole clay stratum in order to obtain the settlement reported in Table 3.6-10.

Time Rate of Consolidation Analysis

Table 3.6-11 presents the estimated settlement following cap placement for each station zone after 30 days and 1 year. The table also lists the estimated time required to reach 95 percent of the total consolidation settlement.

To estimate the time required for consolidation settlement, the one-dimensional theory of consolidation was used and a correction factor was applied to account for three-dimensional effects. Conventional solutions to the consolidation equation employ non-dimensional variables U, the average degree of consolidation and T_v, the dimensionless time factor, which is defined as follows:

$$T_v = (t*c_v)/H^2 \qquad (3)$$

where T_v = dimensionless time factor

t = time

c_v = coefficient of vertical consolidation

H = drainage height

Analytical approximations to the solution of the one-dimensional consolidation equation were used to relate U with T_v (Holtz and Kovacs, 1981).

For each station zone, the following methodology was used to estimate the amount of settlement after a specific period of time:

1. Evaluation of appropriate coefficient of consolidation and boundary drainage conditions.
2. Calculation of the dimensionless time factor, T_v.
3. Use of charts to obtain average degree of one-dimensional consolidation, U.
4. Calculation of settlement for one-dimensional consolidation.
5. Application of a correction factor to account for three-dimensional effects.

Because consolidation would occur in both the recompression and virgin zones, a range of coefficients of consolidation were derived from both the recompression and virgin zones. The coefficients of consolidation were derived from consolidation tests and are listed in Table 3.6-2. Discussion of the techniques used to obtain these parameters is presented in Section 3.6.3.2.

Equivalent values representing each segment with one drainage height (H_{dr}), and one C_v value were determined using a procedure presented in the U.S. Department Navy, 1982, DN 7.1-235. Each profile was assumed to undergo single drainage, with bedrock located at the bottom.

To account for three-dimensional effects, correction factors were obtained from charts based on solutions of the diffusion theory of consolidation equations (Ladd, 1992; Poulos and Davis, 1972). The time factor values for the percent of consolidation ranging from 5 to 95% were applied to the conventional Terzagni one-dimensional consolidation theory to determine the time rates.

The estimated consolidation settlements after 1 year and the estimated amounts of time required to reach 95 percent consolidation are presented in Table 3.6-11. The large amount of variability in these time rate estimates is due to uncertainty in the exact boundary drainage conditions and uncertainty in the coefficient of consolidation.

3.6.6 Chemical Flux Modeling

3.6.6.1 Description of Model

A chemical flux model developed specifically to predict the effectiveness of chemical containment by capping was used in this evaluation. The model was developed by Dr. Danny Reible of Louisiana State University, for the U.S. Environmental Protection Agency (USEPA) and U.S. Army Corps of Engineers (USACE) (Palermo et al., 1998). It is an analytical model, which predicts the chemical flux into and through a soft sediment cover.

Flux is the rate of flow of material per unit area over time and is analogous to the flow volume per unit area in fluid flow calculations. Diffusive flux is the result of chemical movement from areas of higher concentrations to lower concentration in order to achieve equilibrium. Advective flux is the upward movement of porewater caused by consolidation of the underlying sediments and the cover material and movement of groundwater. The model considers both diffusive and advective fluxes, the thickness of sediment layers, physical properties of the sediments, concentrations of contaminants in the sediments, and other parameters.

A schematic diagram of the model is shown on Figure 3.6-4. Movement of contaminants through a cap is controlled primarily by the effective thickness of the cap and the sorption capacity of the cap material. The effective thickness of cap (L_{eff}) is the cap placement thickness (L_0) minus the thickness of the layer subject to bioturbation (L_{bio}), minus the thickness of the layer subject to compromise by porewater expression from consolidating sediment below ($\Delta L_{sed,A}$), and minus the consolidation distance of the cap (ΔL_{cap}).

The depth of the bioturbation layer takes into consideration the normal life-cycle activities of benthic organisms and plants in the aquatic and marsh environment. The modeling assumes 18 inches for both wetland areas and upland areas. These depths are the depths of plant roots and the activity zones for benthic organisms.

When a soft sediment cap is placed, the sediment of the cap tends to consolidate and the sediment below the cap consolidates due to the weight of the cap. Consolidation typically occurs on a time scale that is rapid compared to the design lifetime of the cap. Consolidation of the cap material reduces the thickness of the cover and the separation between contaminants and the protected environment, while consolidation of the underlying sediment results in the vertical movement of potentially contaminated porewater. However, in addition to reducing the thickness of a cover, consolidation also reduces both the porosity and permeability of a cover, resulting in a reduction in chemical migration rates by both advection and diffusion (Palermo et al., 1998).

Sorption capacity is the ability of the cap material to adsorb and retain contaminants, thus removing them from the porewater that is expressed from the underlying contaminated sediment. An adsorption test was performed on the potential cap material (Section 3.6.4.4), and site-specific adsorption coefficients for copper and zinc were selected. Based upon the adsorption constants, a linear, reversible partition coefficient, corresponding to the highest porewater concentration in the underlying sediment for each segment, was calculated and used in the chemical flux model.

The two times of interest in evaluation of cap effectiveness are the number of years until 5 percent of the steady state flux (T_b) and 95 percent of the steady state flux (T_{ss}) reach the bottom of the bioturbation layer. The time until 5 percent flux is used to represent the time at which chemical breakthrough first occurs at the bioturbation layer. The time until 95 percent flux conditions represents the time at which steady state conditions have been established. The modeling approach assumes that as a result of bioturbation, any chemical reaching the base of the bioturbation layer will instantaneously mix through the entire layer. For this cap design, the minimum required cap thickness for a cap lifetime of a 100 years was also modeled.

Contaminant migration is driven by groundwater seepage and diffusion. The cap will retain the contamination in the groundwater seeping into the subsurface of the cap from below and laterally, or through the edge. Engineering controls for potentially contaminated surface water over the top of the cap will be addressed in the final design. The model results are presented in Section 3.6.6.3.

3.6.6.2 Input Parameters

This section describes the parameters used as input for the model, sources of data, and assumptions. Because various sections of the Slough have different depths available to accommodate backfill and cap material and different range of contaminant concentrations, it was necessary to model the different sections of the Slough separately. Based on habitat characteristics, slough depth, and contaminant concentration, the Slough was divided into six segments.

Key input parameters for the model are summarized in Table 3.6-12. The top of the table presents site properties, including associated groundwater monitoring wells and groundwater conditions, intended land usage after completion of the cap, initial cap thickness, assumed bioturbation depths, physical and chemical properties, and data sources. The bottom of the table presents physical and chemical properties of the potential cap material. The cap material values were determined based on the two composite (2 to 5 feet) sediment samples collected from the new alignment area.

Because the input parameters for the model are similar for Segments 3 and 4, these two segments were combined as one segment from Station 24+00 to 35+00. The chemical flux modeling was conducted for each of the five segments.

When required input parameters (porosity and porewater concentrations) were not available for certain segments, the data from other segments were selected based on the proximity of sample locations and sediment concentrations. The horizontal seepage and the groundwater concentrations for each segment were based on data measured in nearby wells.

The key assumptions of the model as applied for this site include:

- Lateral homogeneity of the underlying sediment and cap material.
- Instantaneous cap and underlying sediment consolidation.
- Uniform consolidation and subsequent porewater expression in the sediment and cap.
- Consolidation of maximum fill-depth (i.e. backfill + initial cap thickness) for each segment of existing Slough.
- Concentrations employed to drive migration in model are observed total porewater concentrations.
- Contaminant concentrations driving migration into the cap are assumed to be the highest porewater concentration measured in the underlying sediment in each segment of the Slough to be capped.
- No depletion of contaminants in the underlying sediment.
- Horizontal contaminant migration by groundwater seepage and diffusion processes only (groundwater seepage is at defined flow rates and assumed laterally homogeneous and constant with time).
- No contaminated surface water flow over the top of the cap.
- Uniform, linear, and reversible sorption into cap materials as defined by separate metal adsorption studies.

In general, worst case analyses were used for each segment. The model results are summarized in Tables 3.6-13 through 3.6-15 and are discussed below.

3.6.6.3 Modeling Results

The results of the chemical flux modeling are presented in terms of the predicted flux of copper and zinc through the cap material for each segment. Table 3.6-13 details the results with the groundwater condition with no vertical seepage. Table 3.6-14 details the results assuming a 0.1 cm/yr vertical groundwater seepage for the site. Table 3.6-15 presents the results with horizontal flow of impacted groundwater into the cap. For comparison purposes, the tables also show the San Francisco Bay Chronic Water Quality Criteria and ERM sediment criteria.

As shown on Tables 3.6-13 and 3.6-14, the effective cap thickness ranged from 0.96 to 1.83 feet. This represents the thickness available for long-term chemical containment once the cap and underlying sediment have consolidated. The breakthrough times (T_b) ranged from 279 years to 337,500 years, meaning it would take a minimum of 279 years for 5 percent of steady state flux to reach the base of the bioturbation layer. It would take more than 1,900 years (T_{ss}) for 95 percent of the steady state flux to reach the bottom of the bioturbation layer. It is generally assumed that a chemical mixes instantaneously through the bioturbation layer once it reaches the bottom of that layer. The tables also indicate the porewater and sediment concentrations at breakthrough time and steady state time (C_b/W_b and C_{ss}/W_{ss}) at the base of the bioturbation layer. The concentrations exceeding criteria are shown in bold. At the breakthrough times, no sediment concentrations exceeded the sediment criteria (ERM). Assuming no vertical groundwater seepage, porewater concentrations exceeded the water criteria for zinc in Segments 3 and 4, and copper in Segment 5 at breakthrough times (Table 3.6-13). Assuming vertical groundwater seepage of 0.1 cm/year, porewater concentration of zinc and copper in Segments 3 and 4, copper in Segment 5 (Table 3.6-14) exceeded water quality criteria at breakthrough times (279 to 26,240 years). However, for a time period of a 100 years, copper and zinc were not predicted to exceed water or sediment criteria.

As indicated by the modeling results, the low vertical groundwater seepage predicted for the site would not affect the effective cap thickness or times to reach steady state. However, seepage would impact contaminant migration and increase the metal flux into the cap layer, resulting in higher porewater and sediment concentrations at the base of the bioturbation layer.

The model also evaluated the effectiveness of the proposed cap thickness (see Table 3.6-12) 30, 50, and 100 years after capping. Model results indicated that no copper or zinc concentrations at the base of the bioturbation layer will exceed water or sediment criteria after 100 years. Furthermore, after 50 years, the model showed no measurable copper or zinc at any of the segments in neither the pore water nor the soil at the bottom of the bioturbation layer. After a 100 years, only very low levels of zinc (0.3 μg/l in porewater and 0.07 mg/kg in sediment) were predicted for segments 3 and 4. These values are well below the San Francisco Bay Chronic Water Quality Criteria (81 μg/l) and ERM sediment criteria (410 mg/kg) for zinc. For all other segments, the model showed no measurable concentrations of copper or zinc neither the pore water nor the soil at the bottom of the bioturbation layer.

Therefore, the model results show that a minimum initial cap thickness of 3 feet (for segments 1,2,5 and 6) and 4 feet (for segments 3 and 4) will effectively isolate measured concentrations of copper and zinc from the aquatic and marsh environment for at least 100 years.

Horizontal groundwater seepage into the cap is not a significant concern due to the low hydraulic conductivity and high degree of sorption in the cap materials. As shown in Table 3.6-15, the potential groundwater impacts to the cap from the west side of the Slough ranged from 0.02 to 7 feet. The Slough section in which the cap would be most impacted by horizontal groundwater flow was in the vicinity of the tide gate (Segments 3 and 4) due to higher seepage rates and COC concentrations in groundwater. The impact widths in these segments were predicted to be 4.5 to 7 feet in 100 years. All other cap segments had very minor impacts due to horizontal groundwater flow (impact widths of 0.02 to 0.7 feet). Therefore, the cap is predicted to function as a low permeability barrier to minimize horizontal migration of potentially contaminated shallow groundwater from the western side of the existing Slough to the eastern side of the existing Slough.

3.6.6.4 Model Uncertainties and Limitations

There is a degree of uncertainty in all parameters used in the modeling. Physical parameters are based on a limited number of site data. However, considerable heterogeneity in actual site conditions is likely. Conservative assumptions, such as high end values for consolidation, were used. The following summarizes some of the model uncertainties and limitations:

- The chemical flux model was developed based on accepted scientific principles and observed diffusion behavior in laboratory studies. Application of the model with field verification is limited. Agencies including EPA and the USACE are conducting further research on this topic, which may lead to further refinements of the model. Lacking substantial application and verification of the model, conservative assumptions were used in the calculations.
- Modeling of the cap material was conducted with the assumption that sediment removed from the new alignment would be used to cap the existing Slough. Samples of this material yielded various model parameters, including grain size, moisture content, density, porosity, consolidation, copper and zinc concentrations. However, the parameter values are based on only two samples, which were composited from depths of 2 to 5 feet. Material from the new alignment could have significant variation in physical properties and chemical concentrations. In addition, if a different sediment source is used for cap material, with different physical and chemical properties, the model results could be significantly different. The material to be used for the cap will be tested prior and during construction in accordance with the plans and specifications to be prepared.
- Movement through a cap is controlled primarily by the sorption capacity of the cap and effective cap thickness. Breakthrough time and solid loading for metals depend significantly upon sorption coefficients. The sorption constant was selected based on a lab test using the sediment sample from the new alignment. A nonlinear Freundlich isotherm was observed in these separate metal adsorption studies such that the partition coefficient between sediment and adjacent porewater would decrease as the sediment concentration increased. In the contaminant migration calculations, however, a linear, reversible partition coefficient corresponding to the highest porewater concentration in the underlying sediment was assumed. This is conservative in that the partition coefficient is estimated to be smaller and the mobile porewater concentration estimated to be larger than would actually occur. The estimated partition coefficient assumes no changes in partitioning over time such as might

occur by changes in redox conditions and/or other changes in the sediment and cap environment.

- Because of the uncertainties (two sample points) in the partition or sorption coefficient for the cap material for metals, there is uncertainty in these predicted times. In addition, if cap material other than from the new alignment is used, the sorption capacity could be significantly different, resulting in a different effective cap thickness. The material to be used for the cap will be tested during construction in accordance with the plans and specifications.
- The predictions of concentrations as a function of time assume a semi-infinite cap thickness. The predictions are valid for times less than or equal to T_b but then may deviate from actual predictions of concentrations at longer times due to the inadequacy of the assumption of a semi-infinite cap. At longer times, it is suggested that the steady state flux be used to evaluate cap effectiveness. Even at steady state, the cap will generally dramatically decrease contaminant flux to the environment over current conditions.

3.6.7 Recommended Conceptual Cap Design

The recommended conceptual cap designs for capping the six segments of the existing Slough are presented in this section.

The recommended conceptual cap designs are based on the following considerations:

- Habitat characteristics
- Slough configuration and depth
- Contaminant concentration (sediments, groundwater, and porewater)
- Results of geotechnical analyses (cap and sub-grade consolidation)
- Results of the chemical flux modeling
- Constructability
- Assuming a 100-year cap life, the model results indicate a minimum initial cap thickness of 4-foot for segments 3 and 4, and a 3-foot thickness for all other segments would be required to contain the measured concentrations of copper and zinc in Slough sediments. In other words, no copper and zinc concentrations at the base of the bioturbation layer were predicted to exceed water or sediment criteria in 100 years. Therefore, a 3-foot/4-foot cap on the Slough would be effective for isolation of the contaminants from the aquatic and marsh environment. Based on all the above considerations, a 3-foot initial cap thickness is recommended for all segments except Segments 3 and 4, which will be capped with 4 feet of sediment. For the purpose of this Revised RDR, in sections 3.6.6.1 through 3.6.6.5 below (and associated figures) we have assumed a certain cap elevation for each of the segments. However, the final cap elevations at each segment may be modified to comply with mitigation requirements or other site conditions.

Additional characterization of the cap material (soil along the new alignment and/or import borrow material) will be required prior to placement. The characterization will include soil classification, placement criteria (dry density, moisture content, and porosity), and chemical tests for wetland creation suitability.

3.6.7.1 Slough Segment 1

This segment covers the southern-most reach of the existing Slough from Station 1+00 to Station 15+00. This segment requires creation of wetlands habitat and hydraulic connection between the adjacent marshes. The design grade elevation for the cap in this segment is +1 feet NGVD.

The recommended conceptual cap design is a 3-foot-thick soft sediment cap shown on Figure 3.6-5.

3.6.7.2 Slough Segment 2

This segment covers the reach of the existing Slough from Station 15+00 to Station 24+00. Creation of wetlands is scheduled; therefore, the design grade elevation for the cap in this segment is +1.5 feet NGVD.

The recommended conceptual cap design is a 3-foot-thick soft sediment cap with appropriate excavation for the cap as shown on Figure 3.6-6. Material from the excavation for the cap and/or materials excavated from other areas as part of this project could be placed under the 3-foot-thick soft sediment cap where the existing Slough depth is greater than 3 feet.

3.6.7.3 Slough Segments 3 and 4

These segments cover the reach of the existing Slough from Station 24+00 to Station 35+00. An existing tide gate structure is located at Station 31+00. Therefore, this reach has been divided into two segments. Slough Segment 3 runs from Station 24+00 to Station 31+00 and Segment 4 runs from 31+00 to Station 35+00. The cap grade elevation is limited due to geotechnical considerations (i.e., excessive sub-grade settlement and impacts to adjacent facilities/structures). To limit settlement, the design elevation for the cap in these segments is +3 feet NGVD in Segment 3 and +4 feet NGVD in Segment 4. Creation of wetlands is not planned for these segments. Drainage of surface water runoff would be provided to maintain minimal vegetation for erosion protection.

The recommended conceptual cap design for both Segments consists of a 4-foot-thick soft sediment cap with cap excavation as shown on Figure 3.6-7. The area below the soft sediment cap and Slough bottom could be filled with material from the excavation for the cap and/or materials excavated from other areas as part of this project.

Due to high contaminant concentrations in the groundwater and seeps between Station 24+00 and 35+00, additional cap protection measures may be required. The additional measures may include installation of infiltration controls and/or capping the seeps.

At the beginning of Slough Segment 3 (Station 24+00) and the end of Segment 4 (Station 35+00), a cutoff wall system is recommended to cutoff potential flows along the existing Slough bottom sediments. The approximate location of the proposed cutoff walls is shown on Figure 3.3-1.

3.6.7.4 Slough Segment 5

This segment covers the reach of the existing Slough from Station 35+00 to Station 47+00. Wetlands will be created in this segment. The cap grade elevation will be +3 feet NGVD.

The recommended conceptual cap design in the segment consists of a 3-foot-thick soft sediment cap with cap excavation as shown on Figure 3.6-8. The area below the soft sediment cap and Slough bottom could be filled with material excavated for the cap and/or materials excavated from other areas as part of this project.

3.6.7.5 Slough Segment 6

This segment covers the reach of the existing Slough from Station 47+00 to the Slough mouth (Station 55+00). This segment requires creation of wetlands, hydraulic connection between adjacent marches, and matching existing shoreline. The cap grade elevation will be +3 feet NGVD to Station 53+00. From Station 53+00 to Station 55+00, a gentle slope matching the existing grade is recommended.

The recommended conceptual cap design in the segment consists of a 3-foot-thick soft sediment cap with cap excavation as shown on Figure 3.6-9. The area below the soft sediment cap and Slough bottom could be filled with material excavated for the cap and/or materials excavated from other areas as part of this project, except from Station 52+00 to the Slough mouth. The conceptual end cap design and creation of sandy beach environment are shown on Figure 3.6-10. The end cap transitions from a soft sediment cap to an AquaBlok liner or other suitable product protected by a sand cover.

To cutoff potential flows along the existing Slough bottom sediments, a cutoff wall system is recommended at approximately Station 52+00. The approximate location of the proposed cutoff wall is shown on Figure 3.3-1.

description (see boring logs and well installation logs in Appendix D-1 and D-2). The deeper wells have a 3.75-inch borehole over the screened interval and a 6-inch borehole for the grout seal. All the wells were constructed with 1-inch diameter schedule 40 PVC well pipe, with machine-cut 0.02-inch slots. Petroleum odor was detected in the soil during the installation of wells TI-1 and T1-2. Shallow wells were screened from 1 to 4.65 feet below ground surface (bgs) and the "deeper" wells were screened from 11 to 15 feet bgs. The shallow wells were screened to measure water levels in the shallow root zone and the silty clay interval below the root zone. The deeper wells were used to indicate water levels in the bay mud below the root zone.

Soil samples between the surface and 1.5 feet bgs were collected from eight of the 15 shallow well locations (5 east and three west of the existing Slough). Hand augering was used to advance the hole to the sampling depth and a slide hammer driven split barrel sampler was used to collect the soil samples. Samples were retained in 3-inch diameter, 6-inch long brass sleeves. For shipment to the laboratory, the ends of the brass sleeves were covered by Teflon ® tape and sealed with plastic end caps. Efforts to collect soil samples from the deeper (i.e., 9 to 13 foot) interval using hand tools failed because of the difficulty in augering a 3-inch borehole to the desired depth and inability to retain the soft soils in the hand sampling tools.

Four deeper soil cores were collected at depths of 9 to 11 feet bgs from hydrogeologic core (HC) borings HC-1 through HC-4 located east of the Slough using an all-terrain drill rig to push thin walled Shelby tubes. Because of the numerous CCMVCD channels and the very soft nature of the soils in the marsh, the drill rig could only travel a few feet off the levee or the road just east of Zinc Hill (see Figure 3.4-1) for locations HC-1 and HC-2. The soil description was logged (see boring logs in Appendix D-2). The Shelby tubes are 3-inch diameter and 30 inches long, but were only pushed 24 inches to avoid compressing the samples. All samples were tested in the laboratory for horizontal hydraulic conductivity, porosity, and dry bulk density. Selected samples were also tested for vertical hydraulic conductivity (see Section 3.4.5).

Observations of water levels during pumping performed for development of the guard wells (GRD) indicated that only GRD-1 has a screened interval in a single hydrogeologic unit, and therefore was suitable for slug tests (H2OGEOL, 2001). Based on these results, slug tests were conducted solely on GRD-1.

The results from this investigation are summarized in the following sections.

3.6.8 Hydrogeology Conditions

Three hydrostratigraphic units have been identified at the project site (H2OGEOL 2001, RWQCB 2001):

- The Water Table Unit: This unit comprises the shallowest saturated zone beneath the site. The unit is most pronounced in the southern portion of the site and is comprised of fill, bay mud and peat. All guard wells and those installed during this investigation are located in this unit. The groundwater gradient in this Unit is discussed in Section 3.4.4.
- The Lower Intermediate/Peat Unit: The unit is irregularly distributed in the alluvium beneath the low-lying portions of the site. This unit is particularly prevalent beneath and adjacent to the former evaporation ponds. The unit comprises lenses of peat and peaty sands or mud deep within the alluvium of the site.
- The Bedrock Unit: This unit is encountered in consolidated and/or cemented material that underlies unconsolidated sediments and outcrops at the site. Some portions of the unit are confined while other portions are unconfined. The slope of the potentiometric surface of the groundwater within the unit is oriented to the southeast beneath the southern half of the project site and north towards Carquinez Strait beneath the northern half of the project site.

The Water Table Unit was the focus of this study. The top 15 feet of soil (maximum depth of borings in the newly installed wells) are part of the Water Table Unit. The upper 4 feet of the soil profile has abundant roots and plant debris. Wide areas of vegetation growing in the Peyton Slough marsh plain sediment deposits have produced (over many years) a shallow, variably permeable “root mat” layer of variable thickness overlying lower permeability bay mud which was encountered up to 4 feet bgs. Beneath the root mat layer on the marsh plain there is an extensive deposit of bay mud, which consists mostly of organic rich, fat silty clay with thin lenses (usually less than 2 feet thick) of peat and fine sandy clay (Appendix D-1 and D-2). The presence of this plant material in the root mat interval likely adds interconnecting macroporous structures that increase the hydraulic conductivity in comparison to a soil matrix without plant debris. For the purpose of this study, the Water Table Unit was sub-divided in two hydrogeologic units: the shallow root zone and the deeper bay mud.

3.6.9 Water Levels and Seasonal Variability

Groundwater elevations are within a few inches of the ground surface in both the North and South Peyton Marshes (Table 3.4-1). East of the Slough, the only areas in which the water table is not at or near the ground surface include the elevated dredge spoil piles and an elevated area of sandy soil in the vicinity of well GRD-1 (Figure 3.4-1). These high water table elevation conditions are not unexpected in wetlands bordering on a bay where the elevation of highest high daily tide equals or exceeds the ground surface elevation by as much as a 1 foot.

Historic water level data were analyzed to evaluate whether seasonal variability in water surface elevations exists at the site. Data for 38 monitoring wells were obtained from groundwater monitoring records (H2OGEOL, 2001) compiled since January 1988. Quarterly water levels were measured most years from 1988 through 1996. Since 1997, water levels have been collected semiannually.

The data collected over the past 10 years were divided into five groups for analysis. Two groups representing conditions near the Slough were divided into wells north of the levee (north Slough) and those south of the levee (south Slough). Wells not located along the Slough were categorized by their previously defined hydrostratigraphic unit (Water Table Unit, Lower Intermediate/Peat Unit or Bedrock Unit). The location of these wells is included in Appendix D-4.

Table 3.4-2 lists historic winter and summer water levels as well as average seasonal variations for each monitoring well. Overall, winter water levels have generally been higher than summer water levels. In addition, most of the highest water levels occurred in the winter of 1998, corresponding to the El Niño rain season.

The north Slough wells had an average seasonal variation of 0.33 feet. Average water levels for these nine wells ranged from 2.63 to 5.26 feet during winter, to 2.57 to 4.96 feet during summer. The seasonal fluctuations varied by year, with wet years (such as 1997 and 1998) having higher winter water levels than drier years. For some wells the higher winter levels during this extreme year extended into the next season.

The south Slough wells had an average seasonal variation of 0.87 feet. Average water levels for these six wells ranged from 0.37 to 4.89 feet during winter, to –0.85 to 4.65 feet during summer. Water levels in the three southernmost monitoring wells (MW-20, MW-8A and MW-25) dropped below the sea level for several summers between 1994 and 1998.

The Water Table Unit wells had an average seasonal variation of 1.64 feet. Average water levels for these 12 wells ranged from 1.20 to 12.74 feet during winter, to 0.29 to 10.43 feet during summer. The highest water levels occurred in MW-30, which is located in the northwest corner of the South Ore Body. MW-23A experienced unusual seasonal fluctuations with summer water levels higher than winter water levels from 1993 to 1998. MW-21 (south of MW-20) and MW-22 (south of the Storm Water Accumulation Pond) fluctuated similarly to the south Slough wells with summer water levels dropping just below the sea level during 1996 and 1997.

The Intermediate Unit wells had an average seasonal variation of 0.46 feet. Average water levels for these five wells ranged from 2.37 to 4.99 feet during winter, to 2.24 to 4.67 feet during summer. MW-64, MW-49 and MW-2B located in the South Ore Body area had at least twice the seasonal fluctuation as MW-50 and MW-36 located in the North Ore Body area.

The Bedrock Unit wells had an average seasonal variation of 1.69 feet. Average water levels for these six wells ranged from 3.82 to 12.97 feet during winter, to 2.61 to 10.32 feet during summer. The highest water levels occurred in MW-28, which is located in the northwest corner of the South Ore Body area adjacent to MW-30 from the Water Table Unit.

3.6.10 Hydraulic Gradient in the Water Table Unit

Water level elevations were measured several times between January 30th and February 11th 2002 to estimate the hydraulic gradient (Table 3.4-1). No systematic trend was observed in the differences in groundwater elevations between the shallow and deep wells except in three well cluster pairs. Data were considered indicative of a vertical gradient only if the variability in head was less than the head difference between the shallow and deep wells. That is, the average water level difference between shallow and deep wells had to be less than the standard deviation of the individual measurements. In some case the difference in water surface elevation between the shallow and deep wells in a cluster was due to the difference in ground surface elevation. Since most of the water surface elevations were near the ground surface these data were not considered indicative of a vertical gradient. With these criteria, the only well cluster pair locations that showed a consistent gradient were MW62 and TI-2 which had a downward gradient and MW4A which had an upward gradient. Well locations MW62 and TI-2 had a downward gradient of 0.055 and 0.079 ft/ft, respectively. Since both of these well locations are located near the Slough, this gradient may represent delayed drainage of water stored in the Slough bank from a previous high tide. Well location MW4A showed an upward groundwater gradient ranging from 0.028 to 0.056 ft/ft. These wells are located west of the Slough at a ground surface about 3 feet above the marsh plain. The vertical gradient results may be influenced by local conditions and are not likely to represent conditions on the marsh plain.

There is no consistent horizontal groundwater gradient based on the water elevation data collected between January 30th, and February 11th of 2002. In the north marsh, groundwater generally flows towards the existing Slough or the closest local drainage ditch on the north marsh, with a gradient of 0.006 feet/feet (ft/ft). On the south marsh, the gradient is unclear because much of the ground is flooded. A gradient of 0.006 ft/ft between the area west of the existing Slough and the new alignment was estimated. Therefore a value of 0.006 ft/ft was used as an overall gradient average between the marsh plain and the new alignment.

The horizontal groundwater gradients from the area west of the existing Slough into the cap (after remediation) were also calculated north and south of the levee, and in the tide gate area using the data collected from wells located west of the existing Slough. In the marsh plain north of the levee, the groundwater gradient was estimated to be 0.0025 ft/ft (well MW-62). In the marsh plain south of the levee, the gradient was estimated at 0.026 ft/ft (well MW-25). In the tide gate area, the groundwater gradient ranged between 0.024 (MW-4A) and 0.048 ft/ft (MW-19).

3.6.11 Soil Hydraulic Properties

Eight shallow (1 to 1.5 feet bgs) and four deeper (9 to 11 feet bgs) soil samples were collected for measurement of the soil hydraulic properties. Laboratory permeameter tests (ASTM Method D5084) were used to estimate the horizontal hydraulic conductivity on all the soil samples and the vertical hydraulic conductivity on 10 of the samples. Porosity and bulk density were measured on all the samples (ASTM Method D854). The results are summarized in Table 3.4-3. The laboratory results are included in Appendix D-3.

In the shallow interval (shallow root zone) the horizontal hydraulic conductivities ranged from 1.2×10^{-7} to 2.4×10^{-4} centimeters per second (cm/s) and vertical hydraulic conductivities ranged from 1.1×10^{-7} to 9.5×10^{-6} cm/s. In the deep interval (deep bay mud) hydraulic conductivities were much lower, with the horizontal hydraulic conductivities ranging from 3.6×10^{-8} to 1.3×10^{-7} cm/s and the vertical hydraulic conductivities ranging from 6.4×10^{-8} to 1.6×10^{-7} cm/s. As expected the lowest hydraulic conductivities from samples collected in the bay mud were measured on samples of the fat marine clays. The three samples from within the bay mud which exhibited higher hydraulic conductivities (i.e., TI-1, MW-3A, and MW-4A) were obtained from more coarse-grained samples (silt and silty sand).

The previously installed guard wells were screened from 3 to 15 feet bgs. Drawdown curves from water level measurements collected after well development had an inflection point at which drawdown rapidly increased except from GRD-1. The early, gradual decline in water level followed by rapid drawdown indicates that these guard wells initially draw water from the more conductive shallow zone, which can produce at a much greater rate when pumped than the deeper interval. When the shallow conductive layer tapped by these wells was dewatered, more rapid drawdown of the water in the well was evident because the lower bay mud interval has such low conductivity that essentially no water is yielded to the well (H2OGEOL, 2001). It was concluded that GRD-1 was the only well with a screen interval in a single hydrogeologic unit (deep bay mud) and was, therefore, suitable for slug tests (H2OGEOL, 2001). Furthermore, being located on a topographic high, well GRD-1 did not have standing water, condition that would interfere with a slug test.

A slug test in GRD-1 was conducted by measuring water levels before and after inserting a Troll® freestanding pressure-transducer/data-logger (Troll®) and after adding a water slug to the well. Forty-eight hours were allowed for the water level in the well to return to static after displacement caused by insertion of the transducer. The slug test was initiated by starting the data logger and immediately pouring water into the well to generate the initial displacement. A displacement device was not used because the Troll® cable did not allow the device to be placed in the 1-inch diameter well. The initial water level displacement was 1.17 feet and water levels were measured for approximately 132 minutes prior to the water levels reaching near pre-slug insertion levels (<0.01 feet difference). The KGS Method slug test analysis method (Hyder et al., 1994; HydroSOLV Inc., 2000b) for unsteady flow was used because it provided the best fit for the plotted data collected from the test. The slug test analysis was done using AQTESOLV® version 3.01 software (HydroSOLV Inc., 2000a and b). The hydraulic conductivity calculated with slug test data was 2.6×10^{-5} cm/s (see Table 3.4-3 and Appendix D-5). The Specific storage (Ss) estimated by this method is very unreliable and it was not reported.

The hydraulic conductivity estimated from the slug test analysis on well GRD-1 is greater than laboratory-measured values for the samples collected from the deeper hydrogeologic unit. The difference may be due to macro-porous structures present in the subsurface near the well which acted as preferential flow pathways. These structures are not represented in the very small volume of the soil core samples. However, since the slug test provides a more conservative estimate of the hydraulic conductivity than most of the laboratory data, it was used in the analysis presented in Section 3.5 Contaminant Transport and 3.6 Cap Design.

3.6.12 Preliminary Tidal Influence Study

A study of the effects of the surface water tidal fluctuation on the water levels on wells was conducted between January 25 and January 29, 2002. The tidal study was conducted by placing transducers in nine wells around the site and one in the Slough. Troll® freestanding pressure-transducer/data-loggers were placed in wells TI-1 shallow, TI-1 deep, TI-2 shallow, TI-2 deep, GRD-0, GRD-1, MW-4, MW-4A, and MW-62 (see Figure 3.4-1). The Troll® used to record water levels in the Slough was placed inside a "stilling" pipe which was a 1-inch diameter, 0.01-inch slot Schedule 40 PVC well casing attached to the bridge north of the existing tide gate. The purpose of the "stilling" pipe is to minimize the effects of wave action on the water levels recorded in the slough. The water levels in the wells were related to ground surface elevations by measuring the depth to water from the top of well casing at the start and termination of the test.

This study was conducted spanning the maximum tidal cycle in one-month period (NOAA/NOS, 2002). The total duration of the tidal study was roughly 91 hours. Because of the water level displacement created after the insertion of the transducer in the well, the first 1,000 minutes of data were disregarded and were not used in the evaluation.

Over the duration of the tidal study (excluding the first 1,000 minutes), the highest high water elevation in the Slough was 4.10 feet NGVD, the average high water was 2.47 feet NGVD, the average low water was –0.06 feet NGVD, and lowest low water was –0.93 feet NGVD. The shape of the tidal curve at lowest low tide suggests that water released from the tide gate at low tide increased the minimum water level in the Slough (Appendix D-6).

Influence during the full range of tidal cycle (i.e., response from highest high water through lowest low water) was only observed in wells TI-1 deep, TI-2 deep, MW-4, MW-4A, and MW-62 (Table 3.4-4; Appendix D-6). The maximum tidal range observed in any of the wells was 0.78 feet in well TI- 1 deep. This is consistent with the very low hydraulic conductivity of soils at the site and the relatively large storativity that is present in an unconfined aquifer such as is present at the site. The magnitude of the tidal effect on a well is proportional to the hydraulic conductivity of saturated material at the water table and inversely proportional to the storativity and distance from the water body to the well (Hsieh et al. 1987, Serfes 1991).

Wells GRD-0, GRD-1, TI-1 shallow, and TI-2 shallow indicated only partial tidal influence, showing tidal response only at the highest high water. These wells showed no tidal response when tidal levels were near mean water levels in the Slough. In the cases of TI-1 shallow and TI-2 shallow, this partial influence is interpreted to occur because the screened interval in these wells is near or above the elevation of all but the highest surface water levels. In the case of GRD-0 and GRD-1, the tidal effect is not likely transmitted laterally through the aquifer, but rather though vertical infiltration when the ground surface near these wells floods during peak tides.

Tidal efficiency and lag time were calculated for each well that responded to the full range of the tidal cycle (i.e. TI-1 deep, TI-2 deep, MW-4, MW-4A, and MW-62). The lag time is the time between when the peak occurs in the channel and when it occurs in the wells. The tidal efficiency is the proportion of tidal range in the well relative to the tidal range in the Slough. The lag times ranged from 7 minutes in TI-2 deep to 91 minutes for MW-4A. A lag time of –6 minutes was reported for MW-4; however, this was determined to be due to a clock error in the

Troll® in this well. The tidal efficiency ranged from 0.6 percent in well MW-4A to 7.8 percent in well TI-1 deep.

Overall, tidal fluctuations in the surface water in the Slough have minor effects in the groundwater elevations in the wells considered for this study and therefore a site-wide tidal influence study is not deemed necessary.

3.7 TIDE GATE RELOCATION

Presently, the existing tide gate allows fresh water to flow downstream during low tidal periods and prohibits tide water from flowing into the south Slough area during high tidal periods. A new tide gate structure serving the same function will be installed along the new alignment where it crosses the existing levee. This section discusses alternative locations considered for the tide gate along the new alignment and presents design related information for the structure.

3.7.1 Existing Tide Gate

The existing tide gate structure was constructed in 1998 as part of the ongoing, multi-agency Shell Marsh Restoration Project led by the Contra Costa Mosquito and Vector Control District (CCMVCD). The existing structure is a reinforced concrete structure approximately 67 feet long by 37 feet wide that is founded with a mat foundation designed to float on bay mud. The tide gate structure is independent of a concrete bridge constructed at the same time that is founded on concrete piles driven to bedrock.

Three 72-inch wide by 54-inch high Waterman Model AF-43F aluminum flap gates and two 72-inch wide by 54-inch high Waterman Nekton Self-Regulating gates are mounted on the central headwall of the existing tide gate structure. Both types of gates were manufactured by Waterman Industries, Inc. of Exeter, California.

The structure was constructed inside a sheetpile cofferdam that was used as an outer form for the concrete work. The structural design incorporated the recessed corrugations of the sheetpile as reinforced concrete columns adjacent to reinforced concrete walls constructed inside the sheetpile. After construction was complete the sheetpile at the upstream and downstream ends of the structure were cutoff to conform to the Peyton Slough channel shape. Trashracks located upstream and downstream of the structure are founded on timber piles.

3.7.2 Design Approach

The relocation of the existing tide gate to the new alignment requires the construction of a new tide gate structure, removal of the gates from the existing structure, installation of the gates on the new gate structure, and closure of the existing structure.

The basic design criteria used for the new tide gate are as follows:

- The new tide gate design incorporates the existing tide gate structural design, and reuses all of the gates from the existing structure.
- The levee is not part of an emergency access route for vehicular traffic. Therefore, the tide gate will be constructed without an associated vehicular bridge.

- The new tide gate structure should be designed to limit total settlement to 6 inches and differential settlements to 3 inches.
- The location of the new structure is to toward the eastern end of the levee.
- The invert elevation at the gate will be –3.5 feet NVGD.
- Electrical power will be required at the new tide gate location.
- Design will be in accordance with the Uniform Building Code (UBC, 1997).

3.7.3 Geotechnical Investigation and Findings

A field exploration program for the tide gate structure was conducted. The program included three test borings located on the eastern portion of the levee and one test pit located at the eastern most end of the levee at the base of Zinc Hill. Soil samples were collected from the borings and laboratory testing was performed to measure index properties, undrained strengths, and consolidation characteristics of the soil. The geotechnical data collected was used to assess the behavior of the underlying sediments for different tide gate location alternatives.

The field investigations and laboratory testing found that foundation conditions for the tide gate structure range from soft, highly compressible bay mud below the levee to bedrock at the base of Zinc Hill. The thickness of bay mud below the levee increases west of Zinc Hill where the bedrock surface slopes at a rate of approximately 2H : 1V. Field investigations and geotechnical recommendations for design of the tide gate structure can be found in Geotechnical Investigations for Peyton Slough Remedial Design With Geotechnical Recommendations For Tide Gate Structure, URS (2002) which is attached to this report (see Appendix F).

3.7.4 Evaluation of Tide Gate Relocation Alternatives

Three locations along the eastern levee were considered for the tide gate during the design phase.

- The first location is approximately 140 feet west of the east end of the levee;
- The second is approximately 40 feet west of the east end of the levee;
- The third is at the east end of the levee, at the base of Zinc Hill.

The three locations were evaluated with the following considerations:

- Foundation type (mat or pile);
- Requirements for widening the levee for construction;
- Magnitude of total and differential settlement;
- Constructability issues; and
- Relative cost.

In this evaluation we assumed a similar structural wall design as the existing tide gate for the first and second locations. That is, a structure located 140 feet west or 40 feet west of the east end of the levee would be designed and constructed in a manner similar to the existing tide gate as previously described.

For the third location, at the east end of the levee, a structure with straight walls without columns was assumed where the excavated surface would be bedrock. Sheetpile support for construction of the west wall will still be required. Construction at this location is simpler using conventional excavation, concrete formwork and reinforcement, and backfilling methods. Protection of the excavation for the structure from encroachment of tidal waters from the north and fresh water from the south could be provided by either a temporary levee, which could be a part of the temporary access road for the new alignment excavation or by a sheetpile cofferdam. A portion of the base of Zinc Hill would require excavation and resloping for the structure and for the channel sections immediately north and south of the structure.

A comparison of the alternatives is shown in Exhibit 3.7.1.

Exhibit 3.7.1
Comparison of Tide Gate Location Alternatives

LOCATION (centerline relative to eastern edge of levee)	Foundation Material	Consideration	Advantage	Disadvantage
First Location - 140 feet west	52 feet to 72 feet of soft bay mud; 10 feet of stiffer clay; bedrock	• Requires levee widening for construction. Additional BCDC permitting required.		√
		• Extensive pile foundation required to minimize total and differential settlement. Pile lengths range from 60 to 80 feet. Substantial additional construction cost.		√
		• Wall design will be the same as existing structure, which is dependent on availability of specified sheetpile. Additional construction cost for sheetpile placement, concrete formwork, and reinforcement placement.		√

LOCATION (centerline relative to eastern edge of levee)	Foundation Material	Consideration	Advantage	Disadvantage
Second Location - 40 feet west	8 feet to 24 feet of soft bay mud; 5 feet of stiffer clay; bedrock	• Requires levee widening for construction. Additional BCDC permitting required.		√
		• Pile foundation required to minimize total and differential settlement. Pile lengths are 10 to 25 feet. Additional construction cost.		√
		• Wall design will be the same as existing structure, which is dependent on availability of specified sheetpile. Additional construction cost for sheetpile placement, concrete formwork, and reinforcement placement.		√
Third Location – At the eastern end of levee	Bedrock	• Requires no levee widening for construction. Can be constructed from the south.	√	
		• Foundation on bedrock is favorable for minimizing total and differential settlement.	√	
		• Construction costs are less due to reduced sheetpile placement, concrete formwork, and reinforcement placement.	√	
		• Some regrading of a portion of Zinc Hill required. Requires additional property from Shore Terminals.		√

Chart 3.7-1 shows that locating the new gate structure at the eastern end of the levee is the best alternative from a permitting, engineering, construction, and cost standpoint. Therefore, it is recommended that the tide gate be located at the east end of the levee on a bedrock foundation.

3.7.5 Conceptual Tide Gate Structure

The new tide gate structure will be designed as an approximately 37-foot wide by 67-foot long rectangular reinforced concrete channel located at the east end of the levee (see Figures 3.7-1 and 3.7-2). The invert elevation of the upstream (south) side of the structure and the gates is at –3.5 feet NGVD. The invert elevation at the downstream (north) side of the channel is –5.0 feet NGVD. The 1.5-foot drop in elevation serves to reduce sedimentation buildup against the gates. The gates will be mounted on a central headwall that is designed with the same dimensions as the existing tide gate structure. Trashracks will be located within the concrete channel upstream and downstream of the central headwall. The top elevation of the trashracks will be set at 5.0 feet NGVD which is 1.8 feet above Mean High High Water (MHHW).

Excavation for the structure requires removal of a portion of the levee and underlying bay mud on the west side of the structure and bedrock on the east side of the structure. To the west, the excavation will be supported by sheetpile driven to bedrock and supported inside the excavation with rakers. The sheetpile will be used as the outer form for the west wall and will remain in place after construction is complete. To the east, bedrock will be excavated with a temporary slope of 1H:1V. After the concrete structure has been constructed, the temporary slope will be backfilled with bay mud compacted to 85 percent (ASTM D1557).

Seepage potential along or under the structure is anticipated to be low due to small head differences from upstream to downstream, length of the seepage path along the structure, and low permeability of levee fill and bay mud on the west side of the structure and levee fill against the east side of the structure.

The structure will be protected against undercutting with a 2-foot cutoff wall at the upstream and downstream ends of the structure. Rip rap protection will be placed at the ends of the walls to prevent erosion of the backfill behind the structure walls.

Field conditions found during excavation may require changes to the foundation design of the structure.

Actuators will be installed to open and close the three flap gates in the relocated tide gate structure. This entails mechanical and electrical modifications and a transmitter and receiver set up in order to remotely control the actuators from Mt. View Sanitary District’s control room or an alternate location to be named within CCMVCD’s control. The locally activated actuators facilitate ease of tide gate management, which would improve the utilization of the gates and help control flooding and drainage of the Peyton Slough Marsh system. According to CCMVCD analysis, the actuators will greatly optimize the management of the Peyton Slough Marsh system including McNabney Marsh.

3.8 ENGINEERING CONTROLS

3.8.1 Introduction

During the implementation of the remedial action at the project site, engineering controls will be installed to protect the new alignment and the cap from contamination. As discussed in Section 3.5, the potential pathways for contaminant migration at the project site are (see Figure 3.5-1):

A. Flow from the Ore Body (groundwater and surface water) toward the cap

B. Runoff from the Rhodia Property toward the cap

C. Groundwater transport from west of the existing Slough through the cap

D. Shallow groundwater transport into the new alignment

E. Deep groundwater transport into the new alignment

F. Erosion of contaminated surface soils.

G. Flow from contaminated areas through the paleo-channels

Section 3.8 addresses pathways A (flow from the ore body), B (runoff from the Rhodia Property) and G (flow from contaminated areas through the paleo-channels) through the use of engineering controls. For this report, the engineering controls are presented in preliminary conceptual form. A more detailed engineering analysis of the surface water runoff and groundwater flows in the area are necessary prior to final design of the engineering controls. The final design of the engineering controls for the Property will incorporate the following objectives:

- Minimize stormwater contact with contaminated surface soils
- Minimize stormwater infiltration into contaminated soils
- Minimize erosion of the capped, existing Slough
- Follow BMP requirements of the General Industrial Storm Water Permit
- Minimize design and construction costs.

Section 3.6 Cap design showed that groundwater transport from the west side of the existing Slough (Pathway C) will be controlled by the cap and will therefore not impact the new alignment or the cap. Section 3.5 addresses pathways D (shallow groundwater), E (deep groundwater), and F (erosion) and demonstrated that these pathways will not impact the new alignment.

3.8.2 Existing Conditions

The Rhodia Martinez facility consists of a sulfuric acid processing plant, wastewater treatment plant, process effluent purification (PEP) plant, office, maintenance and warehouse area, stormwater collection system, and undeveloped areas. The storm water collection system drains to the wastewater treatment plant for treatment and eventual discharge to the Carquinez Strait via outfall E001. Rhodia discharge is regulated under NPDES permit (order no. 98-104, permit no. CA0006165) dated October 28, 1998. The NPDES permit regulates the discharges of treated effluent from the on-site treatment plant to the Carquinez Strait (outfall E001) and untreated stormwater runoff to Peyton Slough (outfall E002). Although Rhodia constructed a new storm water outfall in May 1998, the NPDES permit refers to an old storm water outfall which was shared with CalTrans.

The Rhodia facility has historically been divided into six waste management units (WMUs). Two of the waste management units; WMU-1 (the north ore body) and WMU-2 (the south ore body) consist of cinder and slag stockpiles that subsided below the groundwater table, into the underlying soft muds. The groundwater that comes in contact with the subsided stockpiles dissolves soluble constituents, forming a leachate. Outside the project site on Rhodia's property a leachate collection system was installed in both cinder/slag bodies in the early 1970s. The PEP plant was constructed in 1989 for groundwater treatment. The PEP plant uses sodium hydroxide and hydrogen peroxide to remove the high levels of metals from the extracted groundwater. The leachate collection system consists of four sumps which collect groundwater and pump the water to the PEP plant. The capacity of the PEP plant is 80-100 to 130 gallons per minute. Rhodia currently treats water at approximately 100 gpm, so the plant is working within its range of capacity. The PEP plant effluent is treated in the wastewater treatment plant and is then discharged to the Carquinez Strait via outfall E001.

The facility operates four surface water impoundments, including the utility/spill control pond, surge pond, settling pond, and stormwater accumulation pond (see Figure 3.8-1). The utility/spill control pond, surge pond, and settling pond are part of the wastewater treatment system. The stormwater accumulation pond is a lined pond used to hold any stormwater that may need treatment. There is a pump in the stormwater accumulation pond which transfers water to the PEP for treatment. Stormwater is never directly discharged to outfall E002 from the stormwater accumulation pond. North of the west half of the storm water accumulation pond is an unlined, seasonally ponded area. Storm water collection pipes drain the undeveloped area east of the maintenance building to the seasonally ponded area. Water from the seasonaly ponded area is also pumped to the PEP plant for treatment, and is never directly discharged to outfall E002.

Former evaporation pond no. 2 was originally used to hold metal-contaminated groundwater and was closed in November 1995 under Board Order No. 91-166 in accordance with the requirements of the Toxic Pits Cleanup Act (see Figure 3.8-1). The closure of the pond consisted of removing and disposing of sediment in the pond, a sythetic liner and the top one to two inches of soil beneath the liner. The remaining soil was tested for compliance with California Code of Regulations concentrations for Surface Impoundment Closure Requirements. The pond was backfilled and graded to meet storm water collection design specifications which included a catch basin on the west side of the pond. The catch basin drained to the CalTrans stormwater pipe which discharged stormwater from former evaporation pond no. 2 and Highway 680 to Peyton Slough. In 1998, Rhodia constructed new maintenance office buildings and a storm water collection system to support the new buildings. The catch basin in former evaporation pond no. 2 was plugged and a new catch basin was installed in the northwest corner of former evaporation pond no. 2. The pond, however, was not regraded to drain to the new catch basin and the middle, west side of the closed pond remains a low spot. Rhodia constructed a new site storm water outfall pipe to support the maintenance office building storm water collection system and to divide their storm water effluent from the CalTrans effluent. The new catch basin in former evaporation pond no. 2 drains to the new Rhodia storm water outfall. The CalTrans storm water effluent pipe remains currently in place; however, Rhodia's storm water no longer drains to it. The remainder of the Rhodia facility is unpaved with no storm water collection systems, except the area surrounding the maintenance office buildings and the warehouses.

3.8.3 Areas of Potential Impact

Some areas of the project site have been identified as areas of potential impact to the cap from surface water runoff and/or infiltration (see Figure 3.8-1). These areas are unpaved and may contain slag and cinders that could contaminate storm water upon contact. The pervious, unpaved surfaces allow rainwater to infiltrate and come in contact with subsided ore bodies. The following areas of the site were evaluated for storm water control options.

- As described above, the south ore body consists of cinder and slag stockpiles that subsided below the groundwater table, into the underlying soft bay mud. The groundwater that comes in contact with the subsided stockpiles dissolves soluble constituents, forming a leachate. The south ore body area contains visible slag and copper carbonate mass on the ground surface. This area appears to drain, via overland flow, toward the Slough and toward former evaporation pond no. 2. The subgrade portions of the ore body also contribute contaminated groundwater to the flow through a shallow near-surface zone made up of cinders/slag and fill. During periods of high rainfall, when groundwater levels are at their highest, this shallow groundwater flow may surface and enter the cap as overland flow south of the tide gate. Rainfall falling on unpaved portions of the project site may potentially contribute to the shallow groundwater flow as well as to overland flow onto the cap.
- The ground surface surrounding the surge pond and the settling pond (west side of the tidegate and east side of the tidgate) has visible pieces of slag and copper carbonate mass. Once the existing Slough is capped, potentially contaminated storm water should be controlled so that it does not run off or infiltrate onto the Slough cap.
- The spill control pond was in part constructed using slag, potentially mixed with cinders. The area between the spill control pond and the Slough may potentially drain toward the cap. Storm water runoff should be controlled in this area so that it does not run off or infiltrate onto the cap.
- The area southwest of the settling pond, southwest of the tide gate, contains a great deal of construction debris, visible slag, copper carbonate masses, and cinders on the ground surface. This area slopes toward the cap and there are no berms to prevent surfacewater runoff from potentially infiltrating the cap.
- The area between the former evaporation pond no. 2 and the existing Slough is regularly ponded with water. There is a berm along the existing Slough at this area; however, this berm is dredge material that will be removed as part of the Peyton Slough remediation project. Surface water and groundwater will likely drain toward the capped Slough once the dredge material is removed.

3.8.4 Preliminary Design of Engineering Controls

The areas identified above and shown in Figure 3.8-1 will be further evaluated and the most appropriate engineering controls will be designed and implemented. Some of the options that will be evaluated included

- Pumping groundwater from the ore body and/or capping the ground surface over the ore body – A water balance in the ore body will evaluate whether groundwater or surface water infiltration is the major contributor of water to the ore body. If groundwater infiltration is the major contributor, pumping rates may be adjusted from the current winter pumping rate in order to reduce the head in the ore body. If surface water infiltration is instead the major contributor, the ore body may be paved to minimize surface water infiltration into the ore body. Or, a combination of these two methods may be used. Additional measures that may be implemented to control daylighting of groundwater at the ore body include berming and regrading the area to facilitate the collection and possibly treatment of surface water from this area at the onsite PEP.
- Grading and/or capping of contaminated surface soils - To minimize the amount of storm water contact with contaminated surface soils and to minimize storm water infiltration into contaminated soils, some areas known to contain cinders/slag or known subsided ore bodies may be paved/capped. Storm water collected from paved/capped areas will be clean, and therefore, can be discharged to adjacent water bodies. Grading and drainage provided on either paved/capped or uncapped areas may also be implemented. Water collected from uncapped areas would be directed to Rhodia's PEP plant for treatment prior to discharge.
- Berming along the capped Slough - Berms or curbs and gutters or other means of runoff and erosion control may be used in certain areas along the perimeter of the existing Slough to control storm water runoff so that it does not impact the clean cap materials. It is anticipated that berms will be used to provide additional containment where grading is insufficient to capture the design surface water flows.
- Relocation of storm water outfalls - The existing Rhodia storm water outfall, E002, discharges storm water, which is collected from the maintenance office buildings and warehouse, to the existing Slough. The existing Slough will be capped, and therefore, the outfall discharge point may need to be relocated. The discharge point can be relocated by extending the existing outfall pipe to the new alignment or by relocating the outfall pipe to discharge to the adjacent wetland south of the Rhodia facility.
- Plugging or removal of unused utilities - The abandoned drainage pipe east of former evaporation pond no. 2 will be removed.
- Former evaporation pond no. 2 improvements – Former evaporation pond no. 2 may be used as a storm water collection basin for collected storm water from paved and unpaved areas of the facility.

3.8.5 Paleo-channels

As described in Section 1.2, some time after the MOCOCO copper smelting operation began, the original meandering slough found in the southern portion of the project site was straightened and redredged to form its current alignment. A review of historic photographs taken between 1928 and the 1960s shows that the original meandering slough continued to function connecting to the re-dredged Slough. When the copper smelting operation ceased, the original meandering slough as well as other low spots on the Property were filled to grade with copper smelter slag and cinders taken from a nearby cinder/slag pile and then covered with imported fill material. Figure 1.2-3 shows the paleo-channel as it existed in 1960, digitized by overlaying an aerial photo onto the present topography. Site surveys indicate that the portion of the paleo-channel on the Property now lies below fill. The remaining portion of the paleo-channel remains visible in a recent aerial photo, but has become very shallow, most likely as a result of natural accretion. Spot elevations indicate that the current depth is between zero and one foot below the adjacent grade, which is flat and generally varies –1 to +1 ft. NGVD. The historic photos also show that a channel, or borrow pit, was dredged immediately south of the levee in order to provide levee material. This pit has also become very shallow, between one and two feet below the adjacent grade to the south. The soil within these portions of the paleo-channels and borrow pit may provide a preferential pathway for groundwater transport.

The objective of the cutoff walls is to reduce the subsurface hydraulic connectivity between the paleo-channels and the cap and/or new alignment. The proposed cutoff wall locations are shown on Figure 3.3-1. Cutoff walls are proposed at the intersections between the paleo-channels and the new alignment (three locations), at the intersection between the paleo-channels and the cap (four locations) and at the borrow pit (one location). The final location of number of the cutoff walls may be modified based on field conditions during construction activities. A description of the alternative cutoff wall is included in Section 4.2 Implementation Elements and Preliminary Construction Sequence.

4.1 OVERVIEW OF IMPLEMENTATION PLAN

This section presents the implementation plan and the soil and waste management plan for the remedial action design. This section also describes some of the wetland mitigation and enhancement activities that will be implemented by this remedial action. For a more detailed description of the wetland mitigation plan, please refer to Section 4.4.

This plan presents the activities in preliminary form. Activities may be adjusted, as necessary, based on the final permit conditions, selected contractor input and the preparation of detailed plans and specifications. The schedule is presented in Section 4.3.

The general assumptions of this implementation plan are:

1. Final permits and authorizations and their associated conditions, will be incorporated into a final implementation plan in the Plans and Specifications to be issued for construction, including methods and time frame for completion of the project.
2. Access to all areas of the project site will be through Mococo Road and the Rhodia property. The execution of this plan may be facilitated if access is granted through the Shore Terminals property. Rhodia will make every effort to secure such access.
3. Activities conducted at the project site will be in accordance with the site-specific Health and Safety Plan (URS, 2002), which addresses the activities listed in this implementation plan.

4.2 IMPLEMENTATION ELEMENTS AND PRELIMINARY CONSTRUCTION SEQUENCE

The following major elements or activities are included in the implementation plan:

- Site preparation and tide gate and temporary road, trestle, or plate system installation
- Removal and disposal of AOC materials
- Excavation in the new alignment
- Transition to new alignment
- Dewatering excavated materials
- Dewatering and capping the existing Slough
- Property grading and paving

To facilitate the reader, a preliminary construction sequence is provided in the Schedule in Exhibit 4.3-1 and a more detailed description of each element follows.

4.2.1 Site Preparation and Tide Gate and Temporary Road/Trestle/Plate Installation

Prior to beginning the construction of the new alignment, the following site preparation activities will be conducted:

- Species surveys, habitat removed and/or trapping and species relocation
- Site clearing

- Installation of temporary crossings

Concurrent to these preparation activities, the tide gate and temporary access roads, trestles, or plates will be installed.

4.2.1.1 Site Clearing

The proposed layout of the remedial action work areas is shown in Figure 2.5-1. An approximate 7-acre staging area and drying pad will be constructed on the upland area within the Rhodia property to the west of the existing Slough, and will be used for equipment staging, access, and staging of excavated sediments and soils.

The ground surface in the staging area and drying pad will be cleared of debris and upland vegetation and graded to facilitate dewatering of excavated soil and collection of decant water. An additional upland area of approximately 2 acres located in the area west of the northern section of the existing Slough may be available for staging and stockpiling soil. This 2-acre area is owned by the California State Lands Commission, and will require clearing of upland brush, grading and potentially filling to provide a stable construction pad on which to stage equipment and materials.

4.2.1.2 Installation of Temporary Slough Crossings

As shown on Figure 2.5-1, it is anticipated that temporary crossings will likely be necessary for accessing the existing Slough and dredge spoil piles. Temporary crossings are also necessary to carry the equipment across the existing Slough to access the eastern portions of the marsh. The alternatives include, but are not limited to, either trestle-supported bridges or large-diameter culvert crossings. Trestle bridges consist of closed-end pipe piles welded to metal frames to support mat roadways. Culvert crossings consist of embankment fills, which incorporate large-diameter pipes, and are installed in the streambed. Both types of crossings will allow for fish passage.

The selection of the type of crossing will depend on the contractor requirements for equipment types and loads, as well as surface conditions at the crossing locations. Temporary crossings and other temporary structures placed in the existing Slough will be removed at the appropriate time in order to complete the capping of the existing Slough.

4.2.1.3 Tide Gate Installation

A new tide gate structure will be constructed at the east end of the existing levee where it abuts Zinc Hill. The tide gate construction will require approximately 5,000 CY of excavation over a 0.15 acre footprint. Materials to be excavated will include the existing levee material, bay mud, and weathered bedrock. The excavation on the west side of the structure will be supported by an internally braced sheetpile wall.

The new tide gate structure will be designed as an nominally 37-foot wide by 67-foot long rectangular reinforced concrete channel (see figure 3.7-1 and 3.7-2). The structure will include a channel-shaped section through the levee and a central headwall on which the three flap gates and two adjustable self-regulating flap gates will be installed. Just prior to the time of transition from the existing Slough to the new alignment, the existing self-regulating flap gates will be removed from the existing tide gate structure and placed into the new tide gate structure in the new alignment.

During closure of the existing tide gate, the Slough will be cutoff from tide water and upstream flows by installing cofferdams north and the south of the tide gate. A bypass system will be used to move upstream water from behind the southern cofferdam to the downstream side of the northern cofferdam. The cofferdams will remain in place to facilitate the transfer of the gates and other reused materials to the new tide gate structure.

Before opening the new alignment to upstream flows, closure of the existing tide gate and transfer of recycled tide gate material to the new tide gate will be accomplished, as follows:

- Isolate the existing tide gate by cutting off Peyton Slough north and south of the gate with cofferdams and a bypass pipe
- Remove the gates from the structure
- Remove trash racks from upstream and downstream of the tide gate. (Timber piles supporting the trash racks will be cutoff below the bottom of the Slough.)
- Close gate openings in the existing tide gate structure with bulkhead
- Leave handrails on the top of the tide gate structure in place

Gates and trash racks will be washed to remove sediment prior to salvaging and relocation to the new tide gate.

4.2.1.4 Temporary Access

The total surface area for temporary roads, trestles, or interlocking plates for vehicle and equipment access in the wetland areas is approximately 5.4 acres. This acreage includes the temporary roads, trestles, or plates placed along the existing Slough, and by-pass roads near the proposed tide gate area. As shown on Figure 2.5-1, an approximately 30-foot wide strip along the east and west side of the existing Slough will be used to place either temporary roads, trestles, or interlocking plates for equipment access. The conceptual design for temporary roads consists of a 2 to 4 foot layer of clean fill over geotextile fabric. The estimated maximum in-situ volume of road fill (assuming a 3-foot thick road) will be 17,200 CY (over 3.8 ac) for the potential roads along the existing Slough. The roads, trestles, or plates will be used to support heavy equipment in the marsh during construction of the new alignment, and will have turnouts for trucks every 500 feet, as practicable and feasible.

The roads, trestles, or plates will be constructed by first removing and stockpiling the dredge spoil piles along the existing Slough. As the dredge spoil piles are removed, the access roads, trestles, or plates will be contemporaneously placed and advanced.

A similar temporary roadway or interlocking plate system, or a trestle, will be required in order to excavate the new alignment in areas where removal will be completed using a land-based excavator. The method of excavation, particularly in the north reach of the new alignment, will depend on the actual field conditions. Other methods include barge-based dredging using a clamshell bucket. For land-based excavation, the area that this roadway, trestle, or plate system will be placed directly on the new alignment, and therefore, is not included as a temporary loss of wetlands or waters, but rather is included as a permanent loss of wetlands and creation of waters (see Section 4.2.3 for further description of that area). The estimated maximum volume of fill for temporary roads in the new alignment will be 12,000 CY (over 2.1 ac) in the northern section within the BCDC jurisdiction, and an approximate maximum 8,800 CY (1.5 ac) in the southern section. In addition, short sections of bypass roadways, trestles, or plates will be placed to route traffic around the tide gate construction area. The volume of fill of these bypass roads, assuming the same road design, will be at most approximately 1,200 CY (0.2 ac). If plates or trestles are used, the fill volume is significantly less than for roads with fill over geofabric.

4.2.2 AOC Removal

AOC removal includes removal of the dredge spoil piles identified as AOC removal material from along both the east and west banks of the existing Slough, and removal of the AOC identified in the “south spread” area. A detailed discussion of the areas to be excavated is provided in Section 3.2. The south spread area is the low-lying area south of the levee, located between the existing Slough and the proposed new alignment, where dredge spoil piles have spread due to erosion. The soil and sediment in the AOC (shown on Figure 3.2-1 through 3.2-5) is characterized by elevated concentrations of copper, zinc, or low pH. Excavation activities for AOC removal will be conducted prior to any other excavation activities, and will be implemented using conventional land-based equipment.

The total estimated maximum volume of AOC material to be removed, including dredge spoil piles and material in the south spread area, is approximately 42,600 CY.

Prior to initiation of the AOC removal activities, a pre-excavation survey will be conducted to establish the original elevations and contours of the area to be excavated. A post-excavation survey will be conducted to verify that the AOC removal goals were achieved (see Section 4.4).

The removed material will be dewatered as required and either reused onsite or hauled off to an offsite disposal facility. The following tests may be conducted on the material for either disposal or reuse:

- Copper, zinc and pH may be analyzed, in addition to previous analysis conducted on the AOC.
- The paint filter test, and additional analytes required by the landfill, will be run for AOC material that will be disposed of offsite.
- Potentially, AOC material may require Waste Extraction Tests or Total Characteristic Leaching Procedure (TCLP) for verification that the materials do not exceed the Soluble Threshold Limit Concentrations (STLCs) or the federal regulatory limit for COCs, respectively.

Soil and sediments from the existing Slough and the dredge spoil piles are classified as Process Waste from the previous copper smelting operation and are exempt from the classification as a hazardous waste, under federal and state regulations per (40 CFR 261.4(b)(7)(i) and Title 22 66261.4(b)(5)(A)(1), and will be classified as Class B mining waste. The material may be disposed of at a Class I or Class II landfill if the material meets the specific landfill requirements.

4.2.2.1 Dredge Spoil Piles

Removal of dredge spoil pile materials along the existing Slough may be conducted concurrent with temporary road, trestle, or interlocking plate installation. Dredge spoil piles on the east and west sides of the Slough will be scraped and placed in the staging area for placement under the Peyton Slough cap or for disposal. Alternately, the dredge spoil piles may be directly placed in the existing alignment prior to capping. Simultaneously, the temporary access roads, trestles, or interlocking plates will be placed advancing toward the north and south extremes of the project site. These temporary roads, trestles, or plates will be used to access the existing Slough for capping and will be removed upon completion of the project in order to create wetland habitat where the AOCs once existed and restore any other wetland habitat temporarily impacted by these temporary roads, trestles, or plates.

Depending on the extent of AOCs, (see Section 3.2), the removal of dredge spoil pile material, in general may include denuded areas, waters, uplands, and wetlands.

4.2.2.2 South Spread Area

Based on the presence of COC concentrations in excess of SSTLs or Peyton Marsh ambient conditions in the south spread area, soil in this area may be excavated to approximately 1 foot below grade before backfilling the area with clean fill to reach the design elevation. The Habitat Mitigation and Monitoring Plan (HMMP) will be developed (under the USACE permit process) to address the elevation, tidal range, and vegetation restoration goals in the south spread area. The final grade elevation objectives are to revegetate the south spread with wetland habitat, and to provide sufficient embankment height along the new alignment. A summary of the vegetation goals has been provided in Section 4.4.

Backfill material of suitable quality to support wetland habitat will be placed in the south spread area. Backfill material used in the south spread area may be tested for quality and suitability, for such parameters as TOC, general chemistry, and grain size distribution.

4.2.3 Excavation in the New Alignment

Figure 3.3-1 shows the proposed new alignment location. Figure 3.3-1 illustrates the cross-sections of the new alignment at the mouth and north and south of the tide gate. The new alignment will be excavated to a nominal elevation of -3.5 feet NGVD from the Carquinez Strait to approximately 200 feet north of Waterfront Road, using either land-based excavation or dredging, or a combination of the two methods. The design top width of the new channel ranges nominally from 31 to 52 feet, with sidewall slopes of 2H:1V, except in the northern most 200 feet in the mouth area where the sidewall slopes will be 4H:1V due to soft soil(see Section 3.3, Hydraulic Design). The northern most 400 to 800 feet (in the mouth) of the new alignment will be excavated by widening the existing Peyton Slough No. 1 located to the east of the existing Slough. The new alignment will provide at least the same hydraulic capacity of the existing Slough, with an additional 20% width increase in the alignment north of the levee from the existing Slough capacity. This 20% increase is part of the project mitigation.

The maximum estimated in-situ volume of the materials to be excavated from the new alignment is 20,700 CY in the north section (within BCDC jurisdiction) and 4,600 CY in the south section, equivalent to a total in-situ maximum volume of 25,300 CY.

The new alignment north of the tide gate will be removed either using a land-based excavator, or a barge-based dredge. The new alignment south of the levee will most likely be removed using a land-based excavator on a temporary road, trestle, or plate system. The new alignment will not be opened up at the southern tie-in location until the site is ready for transition.

Prior to excavation of the new alignment, the following activities will be conducted:

- Placement of cofferdams in tributaries to the existing Slough that cross the new alignment
- Placement of cutoff walls across paleo-channels that intersect new alignment

4.2.3.1 Cofferdams in Tributaries to the Existing Slough

Cofferdams to isolate flows from the existing Slough toward the vicinity of the new alignment will be installed prior to commencing excavation in the new alignment as shown on Figure 3.3-1. Cofferdams will consist of low permeability soil placed to grade in the four locations of the tributaries that feed the existing Slough. The cofferdams will be placed at low tide to minimize the number of fish that will be present in the tributaries when they are closed off from the Slough. Immediately following placement of the cofferdams, any fish that are caught behind the cofferdams will be removed by seining or some other equivalent method, and relocated to adjacent, species-specific habitat near the project site.

4.2.3.2 Cutoff Walls along New Alignment

Cutoff walls will be placed along the new alignment in order to separate oxbows in the paleo-channel from the new alignment. Cutoff walls are required because the paleo-channel may be potential pathways for contaminated groundwater migration into the new alignment, once the new alignment is excavated.

As shown on Figure 3.3-1, three cutoff walls along the new alignment are anticipated: one 190-foot-long wall where the new alignment parallels the oxbow, and two shorter 30-foot-long walls where the oxbow intersects the new alignment. The cutoff walls may consist of a trench excavated into the impermeable bay mud deposits. The trench will be backfilled with low permeability materials, such as a soil/bentonite slurry wall or clean, fat clay. An alternative cutoff wall design (such as PVC sheetpile walls) may be used depending on the location and associated soil and groundwater conditions. The cutoff walls will be installed from the temporary access roads, trestles, or plates placed on the new alignment, using land-based equipment. Other design or construction specifications may be deemed necessary during preparation of plans and specifications.

4.2.4 Transition from Existing Slough to New Alignment

The diversion of flow to the new alignment will require the use of a diversion dam to redirect the flows from the existing Slough into the newly excavated channel for conveyance of flows from upstream of the project area to Carquinez Strait. Prior to completing the final few feet of new alignment excavation at the tie-in to the existing Slough, a diversion dam will be constructed across the majority of the existing Slough. The diversion dam will be located immediately downstream (north) of the tie-in location. A small notch will be left in the diversion dam and flows from the upstream areas to Carquinez Strait will pass through this notch until the time the diversion into the new alignment is to occur. At that time, the notch in the diversion dam will be filled, and the final few feet of new alignment excavation at the tie-in will be completed. Filling of the notch and completion of the excavation will be timed to occur simultaneously, so that the diversion at the tie-in will be quickly accomplished.

If land-based excavation is used in the north reach of the new alignment, dredging of the mouth of the new alignment will be completed to allow the new alignment to fill in with water prior to completing the diversion of flow. This allows for minimal interruption during the transition of water flows to the new alignment. [Note: If barge-based dredging is conducted, this step will already have been completed]. In addition, the transition will be implemented during high tide to further reduce impacts of the diversion. Once the transition is completed, the diversion dam across the existing Slough will remain in place, and it will then be used as a cofferdam to facilitate dewatering and capping of the existing Slough.

4.2.4.1 Fish Removal in the Existing Slough

Immediately following placement of the cofferdam at the tie-in, the existing tide gate will be closed and the existing Slough will be seined to remove, and relocate fish to adjacent, species-specific habitat near the project site. Due to the expected difficulty in seining the long stretches of the existing Slough, excess water may need to be removed and barriers placed (such as section nets or cofferdams) to localize seining. Fish species will be evaluated upon catching and will be moved to an appropriate habitat near the project site, prior to commencing the capping of the existing Slough. As part of the final permit conditions, a mitigation plan will be provided that indicates the location of available habitat for relocation of fish. A licensed fisheries biologist will prepare the mitigation plan, identify suitable habitat and will supervise the capture, handling, and release of the fish. Cofferdams used to section off the existing Slough into segments will remain in place during the capping of the existing Slough.

The free water remaining in the existing Slough prior to capping will be removed and any additional tributaries to those described in Section 4.2.3.1 will be isolated using additional cofferdams. Additional cofferdams may be required to eliminate newly identified potential hydraulic conduits between the new alignment and the tributaries connected to the existing Slough. These additional cofferdams will consist of low permeability soil placed to grade in identified tributaries, as described in Section 4.2.3.1.

4.2.4.2 Dredging of the Mouth of the New Alignment

Currently, the mouth of the new alignment (located in Peyton Slough No. 1) is approximately 25 to 30 feet wide and has a shallow bottom. This small slough runs south into the marsh approximately 2,000 feet, where its width diminishes to less than 5 feet wide. A sand bar currently exists off the mouth of Peyton Slough No. 1 and will require widening and deepening for the new alignment to function properly. The mouth of the new alignment will be widened and deepened using a barge-based clamshell bucket working from the mouth. The dredging will extend from approximately 225 feet off the mouth into the Strait. Dredging may also extend into the Peyton Slough No. 1 approximately 400 to 800 feet, or as far as necessary to open the new alignment. Approximately 1,800 CY of sediment will be removed from the Strait in order to open the new alignment and the existing Slough for barge traffic. Dredging will continue to the tide gate location if the barge-based dredging option for the north new alignment is selected. If the land-based excavation method is used, the block of material left in place, or the cofferdam placed at the north end of the northern temporary access road, trestle, or plates will be removed using a barge-mounted crane.

[Note: The following will apply, as necessary, if barge-based dredging is used to open the north reach of the new alignment.] If required, a silt curtain or other method may be used to control total suspended sediment (TSS) concentrations during dredging activities. Sediment and soil dredged from the mouth will be placed in a working barge equipped with a barge-mounted crane. Free water will be continuously decanted from the sediments on the working barge. The working barge will transport the dredged material on to the mouth of the existing Slough, where it will be transferred by the barge-based crane into trucks on the temporary access road, trestle, or plates along the west bank of the existing Slough. The material will be stockpiled in the staging area for further dewatering in accordance with the materials handling protocols previously described.

4.2.5 Dewatering of Excavated Material

In the staging area, the sediments dredged from the mouth of the new alignment and the soil excavated from the new alignment will be placed in staging cells where remaining free water will be continuously collected and transferred to storage or settling tanks on the site. The storage tanks will be secondary contained with berms to control spillage. Decanted water will be tested for COCs, and characterized for appropriate disposal and/or reuse on the Site, in accordance with federal, state, and local regulations. Based upon the results, the water may be treated at Rhodia's onsite process effluent purification plant (PEP).

4.2.6 Capping of Existing Slough

Once the flow has been transitioned to the new alignment and the existing Slough has been seined as described in Section 4.2.4.1, the existing Slough will be dewatered and the vegetative layer will be removed in order to place the cap. The existing Slough will be capped, using land-based excavators working from the temporary access roads, trestles, or interlocking plates. The cap will provide physical and chemical isolation of the deeper sediments containing copper and zinc from the aquatic and marsh environment.

The following describes the activities required to commence cap placement in the existing Slough.

4.2.6.1 Placement of Cofferdam and Dewatering of Existing Slough

The existing Slough will be closed by installation of either a cofferdam or a silt curtain equipped with a weir or flapper gate at the mouth of the existing Slough. Dewatering is required to lower the water column, and therefore, minimize the potential for entrainment of contaminated sediments from the bottom of the Slough in the clean cap materials.

4.2.6.2 Capping the Existing Slough

As shown in Figure 3.6-1, the existing Slough has been segregated into six separate sections based on concentrations of COCs, surrounding habitat type, final habitat creation goals, and hydrologic considerations. The analysis of the cap design is discussed in Section 3.6. The soil to be used as cap material will be tested for geotechnical and chemical suitability as described in Section 3.6. The final design basis for each segment (see Figure 3.6-1) is, as follows:

Exhibit 4.2-1
Cap Segments

Segment	Description	Station
Segment 1	Segment 1 is the southern-most segment of the cap, and will require creation of wetland habitat on the cap surface. Design cap thickness in this segment is three feet. The preliminary design grade elevation is approximately +1 ft NGVD, although this grade may be modified based on habitat creation requirements.	1+00 and 15+00
Segment 2	This segment will have wetland habitat created on the surface of the cap, and will be built to approximately +1.5 ft NGVD. Design cap thickness in this segment is three feet. This grade may be modified based on habitat creation requirements.	15+00 and 24+00
Segments 3 and 4	These sections are located just to the north and south of the tide gate, and will be capped with no wetland habitat on the surface. This section is limited in height only by the geotechnical considerations, which prohibit cap construction due to potentially excessive settlement. The adjacent property is Rhodia's operating facility including the polishing pond to the east, and access roads and the south ore body to the west of Segment 3 and the surge pond to the west of Segment 4. Design cap thickness in this segment is four feet. The preliminary design grade elevations are approximately +3 ft NGVD in Segment 3 and up to +4 ft NGVD in Segment 4.	Segment 3: 24+00 and 31+00 Segment 4: 31+00 and 35+00
Segment 5	This segment will have wetland habitat created on the surface of the cap, and will be built to surrounding grade. Design cap thickness in this segment is three feet. The preliminary design grade elevation is approximately +3 ft NGVD. This grade may be modified based on habitat creation requirements.	35+00 and 47+00
Segment 6	The northern-most (Segment 6) segment will have creation of wetland habitat on the cap surface, or at least may not be higher than the surrounding grade in order to promote a hydraulic connection between the adjacent marshes. Design cap thickness in this segment is three feet. The preliminary design grade elevation is approximately +3 ft NGVD, although this grade may be modified based on habitat creation requirements. The design grade tapers at the mouth to match slop in nearby shoreline from stations 53+00 to 55+00. The mouth will be capped and restored with similar characteristics as the surrounding shoreline. To provide added protection against erosion, the cap at the mouth will include a layer of Aquablok™ covered with sand.	47+00 and 55+00

The basic engineering analysis of the cap design indicates that the maximum thickness of cap required for isolation of COCs is 3 feet for segments 1, 2, 5, and 6, and 4 feet for segments 3 and 4. The cap will consist of low permeability soil or bay mud placed over the bottom sediments in the existing Slough and any dredge spoils which may be placed at the bottom of the Slough prior to capping. Based on the design described above, the total volume of cap material to be placed in the existing Slough is approximately 28,100 CY (in-situ). As part of the clearing and preparation for cap placement, a total of approximately 150 CY in-situ of vegetation layer will be removed from within the area to be capped. The excavated vegetative layer will be handled as AOC material.

In addition, the CalTrans storm water discharge pipe and other pipes that currently discharge into the southern section of Peyton Slough will be removed and relocated to wetland areas to the east of the existing Slough or to the new alignment, as practical and feasible.

As shown on Figure 3.3-1, cutoff walls will be installed at four locations where the paleo-channel intersects the existing Slough, and at the intersection of the levee borrow pit and the existing Slough (see Section 3.8.5). In addition, subcap cutoff walls will be placed perpendicular to and in the Slough prior to capping at approximately stations 24+00, 35+00, and 52+00 to minimize the potential for migration COCs in groundwater along the former Slough bed, and into Carquinez Strait (see Section 3.6, Cap Design).

The cutoff walls may consist of a trench excavated into the impermeable bay mud deposits. The trench will be backfilled with low permeability materials such as a soil/bentonite slurry wall or clean, fat clay. An alternative cutoff wall design (such as PVC sheetpile walls) may be used depending on the location and associated soil and groundwater conditions. The cutoff walls will be installed from the temporary access roads, trestles, or plates placed along the existing Slough, using land-based equipment. Other design or construction specifications may be deemed necessary during preparation of plans and specifications.

4.2.7 Site Grading and Paving

Site grading and paving activities will be conducted after remediation activities have been completed, and likely concurrent with restoration activities (see Section 4.4 below). As discussed in Section 3.8, engineering controls will be implemented on the Rhodia Property in order to protect the cap and new alignment from contaminated surface water and groundwater. These controls will require the redirection of water away from the cap, and collection, testing, potentially treatment, and discharge of surface water. Grading activities may require additional fill to raise the ground surface elevation. Surface paving may be implemented to reduce groundwater infiltration in selected areas.

4.3 SCHEDULE

The schedule is provided in Exhibits 4.3-1 through 4.3-4, and represents the sequenced implementation plan described Sections 4.1 and 4.2. The duration and implementation approach for the majority of these activities have been evaluated based on an engineering analysis of cut and fill quantities, material flow utilization, a realistic analysis of existing conditions, and previous construction experience. Because of the complexity of this project, the need to minimize wetland impacts, the size of the AOC, and the sequencing required for quality assurance, this project requires a minimum two-season construction period.

There are several factors that could potential affect the start date. Should there be any significant delays in the commencement of construction, the amount of work that could be completed in the first construction season may be limited to the extreme that the project could be delayed for one entire year to accommodate construction activities for the first season, and to allow the transition to the new alignment to take place before the winter shutdown.

The following assumptions were used in developing this schedule:

- Prior to breeding and nesting season for sensitive species at the project site, surveys will be performed for clapper rail, black rail, and potentially the salt marsh harvest mouse (mouse). Habitat removal may be conducted, and/or trapping out the mouse, to avoid impacts to these species during construction.
- Prior to construction, surveys will be conducted for the red-legged frog (RLF). Presence is not likely. However, if encountered, avoidance mitigation procedures approved by USFWS will be implemented.
- Permits must be received at least 10 weeks prior to start of wetland construction to allow for mitigation of sensitive species, and mobilization activities of the contractor.
- Work is sequenced to minimize the potential for cross-contamination between AOCs and the new alignment.
- Construction activities will be completed utilizing a six-day workweek, 10-hour days in the summer, and 8-hour days in the winter.
- The 7-acre staging area is sufficient to store all materials (imported and exported soil), equipment and supplies without delaying construction.
- Access from the Rhodia property only.
- A suitable source of import bay mud and other imported materials are readily available.
- An operating slough must be functioning continuously throughout the execution of the remedial action. Therefore, the new alignment must be fully operational before the process of closing the existing Slough may begin. *(Note: An implementation option that has the potential to reduce the overall project schedule is the installation of a bypass pipe that will allow for closing the existing Slough and contemporaneously excavating the new alignment while closing and capping the existing Slough. If permitted, the bypass option may be implemented.)*

4.4 WETLAND MITIGATION

This section outlines the approach and the basic components of the comprehensive mitigation plan for impacts associated with the Peyton Slough Remediation Project. Impacts to wetlands, special status species, and their associated habitats have been evaluated (URS 2002). Avoidance, minimization and mitigation measures are proposed to address these impacts.

4.4.1 Wetland Delineation Summary

A wetland delineation was performed to quantify the surface area of waters of the United States located within the project boundary. Waters of the U.S. include both water (tidal or nontidal water) and wetlands. The delineation defines boundaries for the water, the wetland, and the upland.

The wetland delineation was conducted by staff biologists in the project area on January 17, 24, and 31, 2002 (URS 2002). The routine, on-site method described in the *Corps of Engineers Wetland Delineation Manual* (Environmental Laboratory 1987) was used. In the absence of human disturbance or unusual circumstances, an area must possess indicators of three parameters to be considered a jurisdictional wetland: (1) hydrophytic vegetation, (2) hydric soils, and (3) wetland hydrology. This is called the three-parameter approach.

Non-wetland waters of the U.S. in the project site include the Carquinez Strait, Peyton Slough, and its tributaries, Peyton Slough No. 1 immediately east of the Slough, and mosquito abatement ditches that intersect the slough. A tide gate structure, located approximately 2,350 feet inland from Carquinez Strait, currently separates the Slough into a tidal and a nontidal reach. Wetlands in the project area include tidal wetlands and seasonal wetlands. Tidal wetlands exist to the north of the tide gate and the levee that runs east and west between the gate structure and Zinc Hill, Figure 1.2-3. Seasonal wetlands occur on land adjacent to the Slough, south of the tide gate and levee.

Mitigation Plan Elements

Section 5 contains a detailed discussion of the regulatory jurisdiction of state and federal agencies. Multiple agencies exert jurisdiction over the waters and wetlands and wildlife that utilize these areas for habitat. Wetland mitigation is required to guarantee no net loss of wetland functions and values (USACE Sections 10 and 404), to compensate for the actual loss of threatened or endangered species habitat (USFWS Section 7), and to satisfy CEQA. A mitigation plan is developed by identifying impacts, planning actions to avoid and minimize impacts, and mitigate for impacts that could not be avoided. The application of this process to the Rhodia remediation project is described below.

Avoidance

As set forth in the Order issued by the Regional Board, Rhodia is required to undertake this project to remediate historical copper and zinc contamination in and adjacent to the Slough between Waterfront Road and Carquinez Strait. Significant portions of the area of concern are located in waters and wetlands. Thus, it is impossible to avoid direct impacts to the waters and wetlands while implementing the Order. Steps will be taken in the project implementation to avoid impacts to special status species. These avoidance steps include:

- Scheduling construction activities to avoid breeding seasons of special status species
- Surveying for clapper rail and red-legged frog to verify non-presence
- Re-routing the new alignment to avoid high quality Salt Marsh Harvest Mouse (SMHM) habitat

Minimization

A team comprised of remedial engineers, hydrologists, and biologists have used an iterative process to develop a remediation design that will function well in terms of contamination containment and hydrology while minimizing impacts to wetlands and waters. Specific examples of design criteria that minimize impacts to wetlands, water, and associated habitats are:

- Sizing the new channel to replicate existing hydraulic capacity, i.e. minimizing wetland loss by not making the channel any wider than necessary.
- Using Peyton Slough No. 1 as mouth of new alignment: This action will minimize wetland loss that would have been associated with dredging an entirely new alignment through the Peyton Marsh plain, and increases the potential for development of additional dendritic channels in the northern portion of the marsh plain.
- Using of areas that will be disturbed (e.g. new alignment and dredge spoil piles) as access roads
- Removal of habitat or trap out SMHM
- Fencing of exclusion area to limit movement of SMHM into the construction area
- Relocating fish from the existing slough to the new alignment when proper hydrologic connectivity has been established
- Restoring all disturbed areas.

Mitigation

The project will result in the temporal loss of wetlands and permanent loss of waters and uplands (i.e. dredge spoil piles). It is anticipated that the project will result in a net wetland gain of approximately 5.2 acres. Loss of waters is anticipated to be approximately 0.3 acres. The following measures are proposed to mitigate for these temporal and permanent losses:

- Wetlands north of the levee have been identified as SMHM habitat. Areas of high quality habitat have been delineated, and all other wetlands north of the levee are consider low quality SMHM habitat. Disturbance to high quality areas will be mitigated for at a ratio of 3:1 (i.e. for 1 acre of wetland disturbed, 3 will be created). Low quality habitat will be mitigated for at a ratio of 2:1. In addition, a 10-foot wide strip of upland on the dredge spoil piles will be mitigated at a 2:1 ratio. The mitigation will be in the form of wetland to account for loss of SMHM refugia.
- Wetlands in the south spread area will be re-graded and enhanced such that they are capable of supporting a community of native wetland vegetation.
- All disturbed and re-graded wetlands will be planted with wetland species such as *Salicornia virginica, Jaumea carnosa, Frankenia salina, Distichlis spicata, Grindelia stricta,* et al.
- Disturbed upland areas, or areas converted to upland, will be seeded with a mixture of native grasses.
- The hydraulic capacity of the slough will be increased by adding 20 percent in width north of the levee, and by clearing obstructions between the railroad culvert and pipelines south of the levee.
- The new alignment will have an increased sinuosity compared to its original design.
- The new tide gate will have remotely actuated automated controls.
- Rhodia Marsh will have enhanced circulation by adding 1st order channels.

- A 10-year monitoring and adaptive management plan including invasive species management will be provided.

The proposed measures may be modified slightly during the permitting process.

4.4.2 Formal Mitigation Plan

A formal mitigation plan will be prepared using the USACE's San Francisco District standard format as issued in *Habitat Mitigation and Monitoring Proposal Guidelines* (USACE, 1991). The detailed mitigation and monitoring plan will be submitted to the USACE after the project plans and specifications and agency consultations are completed. The content outline for the mitigation plan is shown below:

- Executive Summary
- Chapter 1 - Project Description
  - Project location
  - Brief summary of overall project
  - Responsible parties
  - Jurisdictional areas to be filled
  - Types, functions and values of jurisdictional areas
- Chapter 2 - Mitigation Goals
  - Habitat types to be created
  - Functions and values of habitats to be created
  - Time lapse
- Chapter 3 - Final Success Criteria
  - Target functions and values
  - Target hydrological regime
  - Target jurisdictional acreage to be created
- Chapter 4 - Proposed Mitigation Site
  - Location and area of mitigation site
  - Ownership status
  - Existing functions and values of mitigation area
  - Present and proposed uses of mitigation area
  - Jurisdictional delineation
  - Present and proposed uses of all adjacent areas
  - Zoning

- Chapter 5 - Implementation Plan
  - Rationale for expecting implementation success
  - Responsible parties
  - Site preparation
  - Planting plan
  - Schedule
  - Irrigation plan
  - As-built conditions
- Chapter 6 - Maintenance During Monitoring Period
  - Maintenance activities
  - Responsibilities
  - Schedule
- Chapter 7 - Monitoring Plan
  - Performance criteria
  - Monitoring methods
  - Annual reports
  - Schedule
- Chapter 8 - Completion of Mitigation
  - Notification of completion
  - Corps confirmation
- Chapter 9 - Contingency Measures
  - Initiating procedures
  - Alternative locations for contingency mitigation
  - Funding mechanism
  - Responsible parties

4.5 LONG TERM MONITORING AND POST-REMEDIAL CONTROLS

4.5.1 Monitoring Objectives

This section provides a description of proposed procedures for performing self-monitoring and evaluating the stability of copper and zinc concentrations in the sediments, surface water, and in the groundwater adjacent to the new alignment. In addition, this section provides the post-remedial controls proposed to evaluate the effectiveness of the remedial action.

The overall goal of the long-term monitoring is to evaluate the ongoing compliance with environmental quality goals in the vicinity of the new alignment. The long-term monitoring plan will evaluate whether copper and zinc concentrations in groundwater, surface water, sediment are stable after source removal. To meet that end, the long-term monitoring plan will focus in the following items:

- Evaluation of whether copper and zinc concentrations in groundwater, sediment, and surface water quality adjacent to the new alignment are stable.
- Evaluation of restoration and provisions for adaptive management of temporarily disturbed habitat. (This is discussed in Section 4.4.)
- Provision of post-remedial controls to further protect the newly created habitat and ensure long-term effectiveness of the remedial action.

In addition to the long-term monitoring, post-remedial controls will be implemented to evaluate the effectiveness of the remedial action and to protect the engineered cap and the new alignment through a series of performance thresholds. The post-remedial controls will include institutional constraints as requested by the RWQCB Order (see Appendix A). Descriptions of the proposed monitoring program and the remedial controls are provided in the following sections.

4.5.2 Groundwater Monitoring

4.5.2.1 Hydrogeology

As described in Section 3.4, three hydrostratigraphic units have been identified at the project site (H2OGEOL 2001; RWQCB 2001): the Water Table Unit, the Intermediate/Peat Unit and the Bedrock Unit. The water quality protection strategy presented herein is limited in scope to water bearing zones that immediately surround the new alignment (the Water Table Unit). As described in Section 3.4, the Water Table Unit comprises the shallowest saturated zone beneath the site and consists of two distinct hydrogeological units. Up to 4 feet bgs of the soil has abundant roots and plant debris. Wide areas of vegetation growing in the Peyton Slough marsh plain sediment deposits have produced (over many years) a shallow, variably permeable “root mat” layer of variable thickness overlying lower permeability bay mud. Beneath the root mat layer on the marsh plain there is an extensive deposit of bay mud. The soil consist mostly of organic-rich, fat silty clay (i.e., bay mud) with thin lenses (usually less than 2 feet thick) of peat and fine sandy clay (Appendix D-1). The presence of plant material in the root mat likely adds interconnecting macroporous structures that increase the hydraulic conductivity in comparison to a soil matrix without plant debris. Groundwater flow in the uppermost water-bearing unit (the Water Table Unit) in the project site is largely controlled by topography, with its motion directed toward the nearest water table flowing into the existing slough channel.

4.5.2.2 Proposed Well Network

The objective of the groundwater monitoring is to verify that copper and zinc concentrations in pore water located within cinder/slag interstices do not cause significant impacts in the new alignment by lateral groundwater migration in the shallow root mat and in the bay mud. In the RA that was performed for Peyton Slough (URS, 2002), a conceptual model was developed to describe groundwater interaction with surface water and benthic/sediment pore water in the new alignment. In addition, a mathematical model was developed to quantitatively evaluate intermedia transfers of copper and zinc between groundwater and surface water, as well as with benthic sediments. Proposed numerical groundwater quality objectives (see Exhibit 4.5-1 below) were developed for consideration in evaluating the stability of copper and zinc concentrations in groundwater.

The planned groundwater monitoring will be accomplished using a combination of existing monitoring wells west of the Slough and a network of new post-remediation monitor wells (PRMW) located between the existing Slough and the new alignment. The new monitoring well network includes a line of paired wells (PRMW 0-8), immediately west of the new alignment.

The proposed location of the well network is shown on Figure 4.5-1. These locations are approximate and may be modified based on field conditions and the final new alignment location. The role of each well, and the rationale for selecting well locations, are described in Exhibit 4.5-2. A network of monitor wells (GRD) was installed in June 2000. Several of these GRD are located in the construction zone, and it is anticipated that these wells will have to be abandoned prior to construction. Furthermore, the GRD wells were screened across both the shallow root mat and the deeper soil. Monitoring of these wells may not be representative of the hydrogeology and chemistry at the project site. Upon completion of construction, the PRMW well network will be installed at the locations shown in Figure 4.5-1.

One pair of wells will be installed at each PRMW location: one well screened in the shallow root mat zone (up to 4 feet bgs) and one in the deeper soil (from 10 to 15 feet bgs). Well locations were selected with the aim of providing representative coverage of the new alignment perimeter while targeting specific locations of potential vulnerability with respect to lateral groundwater migration. Therefore, the locations of the PRMW wells will provide two mechanisms that will protect against recontamination of the new alignment: (1) from either shallow or deeper groundwater, and (2) by strategically placing wells in the locations of most likely migration.

Exhibit 4.5-1
Rationale for Monitoring Well Placement in Peyton Slough

Upland MW Wells	URS Proposed Well Pairs (03/2002)	Currently Installed Wells (2/2002)	Rationale for Monitoring Wells' Placement in Peyton Slough
62, 51	PRMW 0	GRD 0	– Near Geoprobe location SSB-1 with elevated zinc – Provide coverage at north end of new alignment
18	PRMW 1,	GRD 1	– Provide coverage in middle portion of north end of new alignment – Opposite the eastern offshoot of the north ore body

Upland MW Wells	URS Proposed Well Pairs (03/2002)	Currently Installed Wells (2/2002)	Rationale for Monitoring Wells' Placement in Peyton Slough
4A, 57, 58	PRMW 2	GRD 2	– Provide coverage in the south portion of the north end of new alignment – Opposite the southern ore body – Opposite paleo-channel near tide gate
57	PRMW 3	GRD 3	– Monitor groundwater downgradient of surge pond – Evaluate shallow groundwater hydrology
19	PRMW 4	GRD 4	– In vicinity of SSB-5 containing elevated copper and zinc – Across slough from MW 19 with elevated zinc – Near existing Slough and dredge spoil piles
3A	PRMW 5	GRD 5	– Provides coverage in middle portion of south end of new alignment – Opposite MW 3A with elevated zinc – Near SSB-6 with slight copper and zinc – Near existing Slough and dredge spoil piles
20	PRMW 6	GRD 6	– Provides coverage in middle portion of south end of new alignment – Near existing Slough and dredge spoil piles
8A	PRMW 7	GRD 7	– Between geoprobe locations SSB-7 and SSB-9 in dredge spoil piles with elevated copper and zinc – Near SSB-8 with slight copper and zinc
25	PRMW 8	GRD 8	– Coverage at south end of new alignment – Opposite MW 25 with elevated zinc in paleo-channel – Near location SSB-11 with elevated zinc

As a means to monitor and evaluate ongoing surface and sediment water quality in the new alignment, site-specific numerical groundwater quality values were developed in the RA (URS, 2002b). Furthermore, since the existing Slough is going to be capped, the groundwater quality values were also to be used for protection of the new alignment.

As described in the RA (URS, 2002b), the numerical groundwater quality values represent copper and zinc concentrations developed to be protective of environmental quality indicators in the new alignment (California Toxics Rule/AWQC for surface water, ER-Ms for sediment). Modeling calculations were performed to account for the following potential mechanisms of transfer between environmental media:

- Lateral groundwater seepage at the aquifer into the new alignment side wall,
- Groundwater seepage into benthic sediment, and
- Pore water exchanges between surface water and benthic sediment interstices.

Site-specific modeling of groundwater/surface water/benthic interactions produced estimates of the maximum groundwater concentrations that could be allowed to seep into the new alignment through sidewalls and benthic underflow without producing an incremental water quality impairment. Incremental water quality impairment is defined as a concentration increase by a margin equal to applicable surface water and sediment quality objectives. Exhibit 4:5-2 summarizes the values developed in the RA.

At the time the site-specific numerical groundwater quality values in the RA were developed, it was thought that the potential source of contamination to groundwater and sediments in the vicinity of the new alignment would be the movement of contaminated groundwater from the vicinity of the ore body, west of the existing Slough, and possibly the existing Slough itself. Modeling performed during the development of the RDR and after the RA was completed suggests that contamination of the new alignment from the movement of contaminated groundwater is highly unlikely. Additional investigations requested by the RWQCB suggest that entire North Peyton Marsh and the wetland areas south of Zinc Hill and east of Peyton Slough may have been impacted by more than 100 years of nearby industrial activity, not related to MOCOCO or its successors. The most noteworthy of these may be the former Peyton Chemical facility located on the eastern flank of Zinc Hill. Peyton Chemical operated a zinc smelter and created at least two slag/cinder piles during its operation. The operation of the smelter and the placement of the associated cinder piles may have contributed to the higher than anticipated levels of metals observed in surface sediment samples collected from North Peyton Marsh.

Therefore, the application of the model-developed numerical criteria to groundwater may not be as appropriate as once thought. A more relevant approach should include: (1) comparison of monitoring results to background groundwater quality, as determined based on wells located in North Peyton Marsh, or other relevant comparison, and (2) for the area south of the levee, time-series analysis to demonstrate that conditions are stable. In any event, monitoring well pairs will be used to track the effectiveness of the remedial action.

The shallow well will monitor the potential seepage of groundwater through the sidewalls of the new alignment. The deep well will primarily monitor the potential upward seepage of groundwater into the new alignment through the benthic underflow. The RA assumed the concentrations from the existing GRD wells (sampled on November 2001) discharged to the new alignment by sidewall groundwater seepage and benthic underflow. The new well configuration will provide a more accurate representation of seepage concentrations into the new alignment.

The proposed numerical groundwater quality goals will be compared to test results for samples obtained from both the shallow and deep wells.

Exhibit 4.5-2
Proposed Numerical Groundwater Quality Goals

Well ID	Dissolved Copper (mg/L)	Dissolved Zinc (mg/L)
PRMW 0	2.2	45
PRMW 1	3.9	51
PRMW 2	5.4	54
PRMW 3	4.8	53
PRMW 4	17	62
PRMW 5	17	62
PRMW 6	17	63
PRMW 7	17	63
PRMW 8	17	63

4.5.3 Compliance Monitoring and Reporting

Several compliance monitoring and reporting tasks will be performed in order to demonstrate the ongoing effectiveness of the remedial design work. Proposed activities and schedules for self-monitoring and reporting are presented individually for (1) groundwater wells, (2) surface water and sediment, and (3) post-remedial environmental controls. A decision tree summarizing monitoring and compliance evaluation activities is presented on Figure 4.5-2.

In addition to the quarterly sampling of PRMW wells listed in Table 4.5-1, the expanded groundwater monitoring program (as amended; URS, 2001) specifies that 11 existing facility wells be monitored. The designations for these facility wells are MW 62, 51, 18, 4A, 57, 58, 19, 3A, 20, 8A, and 25 (see Exhibit 4.5-1).

To establish a baseline of seasonal trends, the quarterly monitoring will be conducted for the first two years and then annually thereafter. If copper or zinc concentrations detected in water samples collected from the PRMW wells demonstrate steadily increasing concentrations of copper or zinc that approach the proposed groundwater quality goals, quarterly sampling of the offending well pair will then be resumed.

Once trends are established by the annual monitoring, a trend analysis of the expanded groundwater monitoring results will be submitted to the RWQCB. If warranted by the trend analysis, a reduction or cessation of the expanded monitoring in the wells may be proposed. In accordance with RWQCB Order 01-094 if exceedances are noted at PRMW wells for four consecutive quarters, then surface water quality monitoring will be required to evaluate whether an exceedance of water quality objectives has occurred in the new alignment. A monitoring plan and reporting schedule will be proposed within 120 days from the confirmation. Surface water and/or sediment quality would be tested in the new alignment adjacent to the well(s) where numerical groundwater quality goals had been exceeded. In addition, a background evaluation would be performed to evaluate copper and zinc concentrations and contributions, if any, from upstream and downstream (tidal) sources.

4.5.4 Post-Remedial Controls

The engineering controls and constructed appurtenances described in this RDR should maintain long-term water quality and sediment quality post-remediation. In addition to the long term monitoring described above, post-remedial controls will be implemented to evaluate the effectiveness of the remedial action through a series of performance objectives. Exhibit 4.5-3 lists the proposed engineering controls and the associated performance objectives.

In order for the planned remediation to remain effective, compliance with the performance objectives listed in Exhibit 4.5-3 should be periodically monitored and appropriate actions should be taken in case of non-compliance.

A dynamic verification monitoring program consisting of periodic visual field surveys will be implemented upon completion of the remedial construction. The dynamic monitoring program will set the performance objectives for future monitoring actions based on the results of previous monitoring events. The field survey program will include visual observations of the cap to monitor the cap conditions and consolidations. These surveys will be conducted in conjunction with wetlands restoration surveys and will be conducted by qualified personnel. Field visual surveys will be generated to indicate changes in conditions from the final construction ground surveys to qualitatively evaluate erosion and changes in the cap surface conditions.

Exhibit 4.5-3
Remedial Performance Objectives

Proposed Engineering Controls	Performance Objectives
Ore Body (OB): (1) Divert surface water runoff away from the cap (see Section 3.8). (2) Continue to pump groundwater out of the south ore body (see Section 3.8).	– Objective OB-1: Minimize surface water contact to the cap and therefore minimize erosion. – Objective OB-2: Continue to prevent groundwater in the south ore body from overflow discharges to the surrounding Rhodia Property or to the cap.
Paleo-Channels (PC) (1) Install cutoff walls in buried cinders/slag paleo-channels (see Sections 3.5 and 3.8)	– Objective PC-1: Eliminate or reduce the flow of impacted groundwater from the buried cinders/slag toward the cap or new alignment.
Existing Slough (ES): (1) Install a low-permeability engineered soil cap to isolate contaminated sediments	– Objective ES-1: Isolate COCs from habitat and prevent contaminant transport through the cap.

in the Slough (see Section 3.6). (2) Install cutoff walls beneath the cap (see Section 3.6).	– Objective ES-2: Prevent longitudinal migration of impacted groundwater through sediment/cap interface to the Strait.
Dredge Spoil Piles (DS): (1) Excavate dredge spoils and impacted soil (see Section 3.2). (2) Backfill clean soil in "south spread area".	– Objective DS-1 & 2: Mitigation of sources by removal, and restoration of habitat.

4.5.5 Institutional Controls

In response to the RWQCB Order's request (see Appendix A) "to identify possible control methods to prevent or minimize human exposure to soil and groundwater contamination within the existing Slough upon completion of the remedial action," Rhodia will develop and implement institutional constraints. The institutional constraints will identify control methods to protect the cap and to regulate the use of the capped area subsequent to the completion of the remedial action to minimize cap disturbance. Rhodia will submit the proposed institutional constraints as required under Task C.6 of the Order.

The institutional constraints will also address the RWQCB requirement to have an enforceable mechanism in place to demonstrate the ongoing compliance with site mitigation conditions. It will provide a list of maintenance procedures and a maintenance schedule that should establish ongoing prevention of the recontamination of the new alignment.

A Covenant and Environmental Restriction will be recorded in the official records of Contra Costa County by Rhodia and potentially the California State Lands Commission, containing the limitations to regulate the use of the remediated area. The Covenant and Environmental Restriction will be incorporated into each deed of any portion of the remediated area and may impose a variety of limitations and conditions, such as limiting access to the capped slough, and/or restriction in drilling operations.

5.1 REGULATORY OVERVIEW

The California Regional Water Quality Control Board, San Francisco Bay Region (RWQCB), is the lead regulatory agency providing oversight for the upcoming environmental remedial actions that will be performed at the Site. There are four permitting agencies in addition to RWQCB that have jurisdictional authority over the proposed action, including: the United States Army Corps of Engineers (the Corps), the San Francisco Bay Conservation and Development Commission (BCDC), the California State Lands Commission (SLC), and the City of Martinez (the City). In addition to obtaining permits from these agencies, the proposed remedial action must be in compliance with the California Environmental Quality Act (CEQA). RWQCB is the CEQA lead agency for the project.

There are seven permits required for the implementation of the selected remedial action alternative including:

- RWQCB: 401 Water Quality Certification under the federal Clean Water Act (CWA)
- Corps: Individual Permit, Sections 10 and 404 of the CWA, which includes consultation with
  - US Fish and Wildlife Service (USFWS): Biological Opinion
  - State Historic Preservation Officer (SHPO): concurrence under Section 106 of the National Historic Preservation Act (NHPA)
- BCDC: Major permit under the McAteer-Petris Act
- SLC: Surface Lease/Dredging Permit
- City:
  - Grading and drainage permit,
  - Structural permit, and
  - Hauling permit.

It should be noted that the Individual Permit process under Sections 10 and 404 of the CWA requires that the Corps consult other federal agencies with jurisdictional authority to regulate resources potentially affected by the proposed action. Therefore, as part of the permitting process with the Corps, the project will need to obtain a Biological Opinion from the USFWS under Section 7 of the federal Endangered Species Act (ESA) and concurrence from the SHPO under Section 106 of the NHPA. The following sections 3.1 through 3.4 summarize the regulatory history, agency requirements, and permits required by each of these agencies.

5.2 REGIONAL WATER QUALITY CONTROL BOARD

In 1949, the Dickey Water Pollution Act created a State Water Pollution Control Board that evolved into the State Water Resources Control Board (State Water Board) in 1967. The California Legislature recognized that problems of water pollution in California vary greatly from region to region. Consequently, the Dickey Water Pollution Act also established nine regional water pollution control boards located in each of the major California watersheds. In 1969, the California Legislature enacted the Porter-Cologne Water Quality Control Act (PCA), also known as the California Water Code, which establishes the regulatory framework for the regulation of waste discharges to both surface and ground waters of the State.

Through the PCA, the State Water Board and the nine regional water quality control boards have been entrusted with broad duties and powers to preserve and enhance all beneficial uses of the state's immensely complex waterscape. Today, the State Water Board and the nine regional boards implement both the PCA and CWA in a coordinated manner. Section 13302 of the PCA authorized the state and regional boards to order any person who has discharged pollutants into the waters of the State of California to take remedial action.

In the San Francisco Bay Area, designated as Region 2, RWQCB conducts planning, permitting, and enforcement activities under the California Water Code. The overall mission of RWQCB is to protect surface and ground waters of the San Francisco Bay Region. In 1997, the RWQCB Bay Protection Toxic Cleanup Program added Peyton Slough to the list of “toxic hot spots”, therefore requiring Rhodia to prepare a remedial action plan to address contamination. The proposed remedial action plan must be approved by RWQCB.

Because the proposed remedial action will require disturbing an identified “toxic hot spot”, all project activities must comply with Section 13396 of the California Water Code. Section 13396 states that “no person shall dredge or otherwise disturb a toxic hot spot site that has been identified and ranked by a Regional Board without first obtaining certification pursuant to Section 401 of the CWA (33 United States Code [USC] Section 1341) or waste discharge requirements.” Therefore, in order to perform the selected remedial alternative, a 401 Water Quality Certification must be obtained from RWQCB.

5.3 ARMY CORPS OF ENGINEERS

Section 404 of the CWA designates jurisdictional authority over “Waters of the United States” to the Corps (33 USC 1344). Waterways subject to the Corps jurisdiction in the San Francisco Bay Area include riparian, seasonal and perennial wetlands, and mudflats found within and alongside waterways and the Bay. The Corps is authorized to issue permits, after notice and opportunity for public hearings, for the discharge of dredged or fill material into waters of the United States at specified disposal sites. These discharges include return water from dredged material disposed on the upland and generally any fill material (e.g., rock, sand, dirt) used to construct land for site development, roadways, and erosion protection.

The Corps also exerts jurisdiction under Section 10 (33 USC 403) of the Rivers and Harbors Acts (1890 and 1899), which covers construction, excavation, or deposition of materials in, over, or under such waters, or any work that would affect the course, location, condition, or capacity of any navigable waters of the United States. Section 10 is the most frequently cited regulation that allows the Corps to exert jurisdictional authority over a proposed action. Projects that involve work in and around the shoreline of San Francisco Bay often fall under both Section 404 and Section 10 jurisdiction.

When the Corps takes jurisdictional authority over a project that involves other federal agencies, the Corps becomes the lead federal regulatory agency (or federal nexus agency) through which the other federal agencies assert their regulatory authority (33 Code of Federal Regulations [CFR] 320). For projects that do not qualify under the Nationwide Permit Program, the Corps is required to consult with the other federal agencies that have jurisdiction. Therefore, when the

Corps issues an individual permit for a project, all federal regulatory requirements must be met. Section 404 of the CWA requires the Corps, when issuing the permit, to follow the requirements of the United States Environmental Protection Agency's (EPA's) guidelines for implementing Section 404(b)(1) of the Clean Water Act (40 CFR 230 et seq.).

The project site is located within the jurisdiction of the San Francisco Corps Regulatory District, and the Corps has deemed this project requires an Individual Permit. The Corps has jurisdictional authority over the proposed project because it would result in actions needing Corps approval under Section 10 and Section 404. As stated above, other federal agencies with jurisdiction over the proposed action include USFWS and SHPO. As part of the application for an Individual Permit, a Biological Assessment (BA) and a Cultural Resources Technical Report have been drafted and will be submitted to the Corps for consultation with these other two federal agencies. The completed application will be submitted to the Corps with the completed BA and Cultural Resources Technical Report concurrent with the submittal of this RDR.

5.4 BAY CONSERVATION AND DEVELOPMENT COMMISSION

Section 66600 of the McAteer-Petris Act (the Act) enabled the California legislation to create BCDC as a response to haphazard and uncoordinated filling of the San Francisco Bay (the Bay). The primary purpose of the Act is to promote responsible planning and regulation of the Bay. The Act emphasizes the elimination of unnecessary placement of fill in the Bay, use of the Bay for water-oriented uses, and the inclusion of public access consistent with a proposed project. BCDC's jurisdiction generally extends to a 100-foot shoreline band of all areas of the Bay that are subject to tidal action, including sloughs and marshlands as well as saltponds and managed wetlands (as defined in the Act) and certain designated waterways.

The Act requires that individuals obtain permits to place fill (pilings, floating structures, boat docks and other solid materials), extract materials (dredge), or make substantial changes in use of land, water, or existing structures in the Bay. In determining whether to issue permits, BCDC refers to policies set forth in the Act and in the San Francisco Bay Plan. In general, these policies authorize fill or excavation of wetlands only for water-dependent projects that lack any feasible upland alternative, and only if wetland impacts are mitigated. Under the Act, BCDC may approve a fill project only if it is demonstrated that the proposed fill is the minimum amount necessary to achieve the remediation of the site (Section 66605(c)), public benefits clearly outweigh any public detriments (Section 66605(a)), the proposed fill contributes to a water-orientated use (Section 66605(a)), or the project is necessary to the health, safety, or welfare of the public in the entire Bay Area (Section 66632(f)). The Act lists the following as examples of water-oriented uses: ports, water-related industry, airports, bridges, wildlife refuges, water-oriented recreation, and public assembly.

BCDC issues four types of permits: major, administrative, emergency, and region-wide. BCDC also grants federal consistency determinations under the Coastal Zone Management Act for areas within its jurisdiction. If the selected remedial alternative is classified by BCDC as a "minor repair or improvement," the project will warrant the issuance of an administrative permit. The review process for an administrative permit is generally short, as it does not require a public hearing. However, if the project is not considered to be a minor repair, a major permit will be required. The review process for a major permit is more extensive and the application may be reviewed at hearings held by the engineers and designers who advise the Commission (BCDC review board). BCDC staff has determined that the project qualifies for a major permit.

5.5 CALIFORNIA STATE LANDS COMISSION

The SLC was established in 1938 with authority detailed in Division 6 of the California Public Resources Code. SLC has broad mandates for protection of California's natural environment, including holding in trust all lands of the state of California for all present and future generations by not allowing development incompatible with uses covered by the Public Trust Doctrine. Historically, the Public Trust Doctrine provided for the public uses of California waterways, which were defined as "commerce, navigation, and fisheries." Later court rulings added hunting, fishing, swimming and recreational boating to the list of public uses, and in 1971 the list was again expanded to include "preservation of those lands in their natural state" in order to protect scenic and wildlife habitat values. Based on a 1983 California Supreme Court ruling [National Audubon Society v. Superior Court, 33 C3rd 419], SLC also has an "affirmative duty to take the public trust into account" in making decisions affecting public trust resources, and also the duty of continuing supervision over these resources which allows and may require modification of such decisions. The Commission follows this mandate when considering the use of "Sovereign Lands" under its jurisdiction, and seeks cooperation of other agencies having authority over public trust resources.

Public and private entities may apply to the Commission for leases or permits on State lands for many purposes including marinas, industrial wharves, tanker anchorages, harvesting of timber, dredging, grazing, mining, oil and gas, or geothermal development. The definition of a SLC lease includes a permit, right-of-way, easement, license, compensatory agreement, or other entitlement of use. SLC maintains broad discretion in all aspects of leasing including the category of lease or permit, the determination of which use, method or amount of rental is most appropriate, whether competitive bidding should be used in awarding a lease, what terms should apply to a lease, how rental should be adjusted during the term of a lease, whether bonding and insurance should be required and in what amounts, and whether an applicant is qualified (based on what it deems to be in the best interest of the State) (6 PRC 2000).

5.6 CITY OF MARTINEZ

A grading and drainage permit is required by the City of Martinez prior to engaging in activities that involve filling or grading of land parcels within the city limits. Prior to submitting the grading permit application to the City's Department of Public Works, Engineering Division, any additional required permits and/or consents from private owners and public agencies (including those with proprietary rights and regulatory jurisdiction) must first be obtained.

In addition, the City requires a structural permit for the tide gate, and also requires that projects involved in the transport of materials through the City limits must obtain a haul permit.

5.7 CALIFORNIA ENVIRONMENTAL QUALITY ACT COMPLIANCE

The basic goal of CEQA (Pub. Res. Code Section 21000 *et seq*.) is to develop and maintain a high-quality environment now and in the future. Specific goals of CEQA are for California's public agencies to:

- Identify the significant environmental impacts of their actions; and either
- Avoid those significant environmental impacts, where feasible; or
- Mitigate those significant environmental impacts, where feasible.

CEQA applies to projects proposed to be undertaken or requiring approval by state and local government agencies. Such projects are defined as activities that have the potential to impose a physical impact on the environment and may include the enactment of zoning ordinances, the issuance of use permits, and the approval of tentative subdivision maps. Projects that require approvals from more than one public agency must have one designated "lead agency" to complete the environmental review process. It is the lead agency's duty to determine if the project is subject to or exempt from CEQA and to perform an Initial Study to identify the environmental impacts of the proposed project to assess whether the identified impacts are significant.

An Initial Study is the first document prepared in the CEQA process after the lead agency has determined that the proposed activity is "a project." A project is any activity which may cause either a direct or reasonably foreseeable indirect physical change in the environment, and which is directly or partially supported, permitted, certified, or otherwise entitled for use by a state or local agency (summarized from CEQA Statutes, Definitions, 21065). The Initial Study is written to assess the potential environmental impacts by the proposed project. The findings of the Initial Study are used to assist the lead agency in determining whether the proposed project will have significant environmental impacts. Based on these findings, the lead agency prepares one of the following documents:

- Negative Declaration – for projects that have no significant environmental impacts;
- Mitigated Negative Declaration – for projects that were found to have significant impacts, but the lead agency has revised the project to avoid or mitigate the impacts; or
- Environmental Impact Report – for projects that have significant impacts that cannot be sufficiently revised with the addition of minor mitigation measures.

As stated above, RWQCB is the CEQA lead agency for the project. The Initial Study was submitted by RWQCB for public comment on September 4, 2002.

ASCE, 1975. Manuals and Reports on Engineering Practice – No. 54. Sedimentation Engineering.

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